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Structural Steelwork Design to B.S.S. 449 (1948).

F. H. ABRAHAMS, A.M.I.Struct.E., M.Inst.W.

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Part 1.

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STRUCTURAL STEELWORK DESIGN TO B.S.S. 449 (1948).

F. H. Abrahams, A.M.I.Struct.E., M.Inst.W.

PART 1.

EXPLANATION OF THE SPECIFICATION.

Issue of the new edition of B.S.S. 449 is an epoch making event in the structural steel world, not merely because it is a new edition, but on account of the radical changes in procedure involved. While of course basic principles of theory remain unchanged except for normal developments resulting from research, the method of application of structural theory and the procedure for design is fundamentally different, calling for an entirely new approach and in certain instances the development of an entirely new technique. In effect this means that ideas formerly current regarding the selection of sections and the build up of composite sections must, in many cases, be completely scrapped if economy in material is to be achieved, and in all cases must be investigated afresh in the light of new requirements.

First point requiring attention is the quality of steel to be employed in a structure. So far as mild steel is concerned, working stresses in B.S.S. 449 (1948) are based on steel in accordance with B.S.S. 15 (1948), the most important feature of which—from the designer's point of view—is that a minimum yield stress is specified. For plates, sections, and flats from $\frac{1}{4}''$ to $\frac{3}{4}''$ thick minimum yield stress is 15·25 tons per square inch, and over $\frac{3}{4}''$ thick 14·75 tons per square inch.

As no guaranteed yield stress was specified in previous editions of B.S.S. 15, it is essential that designs to B.S.S. 449 (1948) should state clearly that material must be in accordance with the requirements of B.S.S. 15 (1948) if maximum economy is to be obtained. All notes and designs following are on this basis.

Permissible Stresses.

For pure tension the maximum direct axial stress on the nett area must not exceed 9 tons per square inch. Previously the allowable maximum axial stress was 8 tons per square inch. The effective nett area for particular cases is mentioned later.

For compressive stresses on uncased struts the maximum stress on the gross sectional area for axial loads must not exceed the values for F_a given in Table 1, in which the slenderness ratio L/r equals the assumed effective length divided by the least radius of gyration. For the assumed relationship between actual length and effective length for various conditions of end restraint reference should be made to Clause 35 of the specification and the related diagrams given in Appendix E, although as many different types or combinations of end restraint to those illustrated are met with in actual practice the designer must to some extent use his own judgment in the matter, always bearing in mind that theoretically effective lengths are shorter than those given in the specification.

The formula on which allowable stresses in Table 1 is based is given in Appendix C of the specification, and compared with the formula in the 1937 edition of the specification gives a slight increase in allowable working stresses.

It has always been recognised that struts adequately cased in concrete are capable of supporting heavier loads than uncased struts. Struts fail by bending rather than by crushing, and as a concrete encasement—adequately reinforced—increases the resistance to bending, cased struts can obviously carry heavier loads than uncased struts. This condition is now officially recognised in the new edition of the specification. Clause 18 gives full information regarding procedure, and it will be noted that the basis of the increased permissible stress, compared with uncased struts, is the higher value allowed for the least radius of gyration. Note must also be taken that the permissible stress for a cased strut must not exceed that for the same strut uncased by more than 50%.

A very great improvement introduced into the new edition of B.S.S. 449 refers to permissible stresses in angle struts. A full description of the derivation of these stresses is given in Clause 18c and in Appendix D, the general intention being to cover for the effect of eccentricity of connections which previously was often the subject of elaborate calculations. Clause 18c also distinguishes between single angle and double angle struts, a very necessary distinction as a double angle strut connected to a gusset placed between the angles is obviously much stronger than two independent single angles of the same section. Permissible stresses for double angle struts with end gussets located between the angles are as given in Table 1 for other struts. Permissible stresses for single angle discontinuous struts, and also for double angle discontinuous struts where both angles are connected to the same side of the gusset are given in Tables 2 and 3.

Bending stresses are treated more fully and much more satisfactorily in the new edition of the specification than was formerly the case. It has in fact long been known that procedure in the

TABLE 1 (See Table 7 in B.S.S. 449).

Permissible Working Stresses in tons per sq. in. of gross section for Struts with Axial Loads—in accordance with B.S.S. 449 (1948).

l/r	F_a Tons/sq. in.	l/r	F_a Tons/sq. in.	l/r	F_a Tons/sq. in.
0	9·00	86	4·82	164	1·98
10	8·51	88	4·72	166	1·94
12	8·41	90	4·62	168	1·90
14	8·32	92	4·52	170	1·86
16	8·22	94	4·42	172	1·82
18	8·13	96	4·33	174	1·79
20	8·03	98	4·23	176	1·75
22	7·93	100	4·13	178	1·72
24	7·83	102	4·04	180	1·68
26	7·74	104	3·95	182	1·65
28	7·64	106	3·85	184	1·62
30	7·54	108	3·76	186	1·58
32	7·44	110	3·67	188	1·55
34	7·35	112	3·59	190	1·52
36	7·25	114	3·51	192	1·49
38	7·16	116	3·42	194	1·47
40	7·06	118	3·34	196	1·44
42	6·96	120	3·26	198	1·42
44	6·86	122	3·19	200	1·39
46	6·77	124	3·11	205	1·33
48	6·67	126	3·04	210	1·27
50	6·57	128	2·96	215	1·22
52	6·47	130	2·89	220	1·17
54	6·38	132	2·83	125	1·13
56	6·28	134	2·76	230	1·08
58	6·19	136	2·70	235	1·04
60	6·09	138	2·63	240	0·99
62	5·99	140	2·57	245	0·96
64	5·89	142	2·52	250	0·92
66	5·80	144	2·46	260	0·87
68	5·70	146	2·41	270	0·81
70	5·60	148	2·35	280	0·76
72	5·50	150	2·30	290	0·70
74	5·41	152	2·25	300	0·65
76	5·31	154	2·20	310	0·62
78	5·22	156	2·16	320	0·59
80	5·12	158	2·11	330	0·55
82	5·02	160	2·06	340	0·52
84	4·92	162	2·02	350	0·49

TABLE 2. (See Table 8 in B.S.S. 449).

Permissible Working Stresses in Tons/sq. in. of gross section for Discontinuous Angle Struts (Double Riveted, Double Bolted, or Welded at Ends) in accordance with B.S.S. 449 (1948).

l/r	F_c2 Tons/sq. in.	l/r	F_c2 Tons/sq. in.	l/r	F_c2 Tons/sq. in.
0	6·00	90	3·40	175	1·54
10	5·71	95	3·26	180	1·47
15	5·56	100	3·12	185	1·41
20	5·42	105	2·98	190	1·35
25	5·27	110	2·85	195	1·30
30	5·13	115	2·73	200	1·25
35	4·98	120	2·60	210	1·16
40	4·84	125	2·48	220	1·08
45	4·69	130	2·37	230	1·00
50	4·55	135	2·26	240	0·93
55	4·40	140	2·15	250	0·86
60	4·26	145	2·05	260	0·81
65	4·11	150	1·95	270	0·76
70	3·97	155	1·86	280	0·72
75	3·82	160	1·77	290	0·67
80	3·68	165	1·69	300	0·62
85	3·54	170	1·61	350	0·47

TABLE 3. (See Table 9 in B.S.S. 449).

Permissible Working Stresses in Tons/sq. in. of gross section for Discontinuous Angle Struts (Single Riveted, or Single Bolted at Ends) in accordance with B.S.S. 449 (1948).

l/r	F_c1 Tons/sq. in.	l/r	F_c1 Tons/sq. in.	l/r	F_c1 Tons/sq. in.
0	3·00	90	1·96	175	1·13
10	2·88	95	1·91	180	1·09
15	2·83	100	1·85	185	1·05
20	2·77	105	1·79	190	1·02
25	2·71	110	1·73	195	0·99
30	2·65	115	1·68	200	0·96
35	2·60	120	1·62	210	0·90
40	2·54	125	1·57	220	0·84
45	2·48	130	1·51	230	0·79
50	2·42	135	1·46	240	0·75
55	2·37	140	1·41	250	0·71
60	2·31	145	1·37	260	0·67
65	2·25	150	1·32	270	0·64
70	2·19	155	1·28	280	0·60
75	2·14	160	1·24	290	0·56
80	2·08	165	1·20	300	0·53
85	2·02	170	1·16	350	0·43

past was both inconsistent and unsatisfactory, and good designers often exercised a wise discretion in the use of previous formulae. For example, a compound beam with a 12″ flange plate had the same permissible stress in bending as the same beam with a 12″ deep channel connected to the flange, yet the channel obviously gives much more lateral stiffness than the plate and therefore justifies a higher working stress.

Tension flanges of beams and compounds may now be stressed to 10 tons per square inch in bending, with a reduction of 5 per cent. in the case of plate girders. The reason for this reduction in allowable stress in plate girders compared with joists and compounds is that the joist fillets give a greater homogeneity of section than is the case with built-up sections. In all cases for tensile stress the nett area of section is to be used in calculating the moment of inertia.

Compressive flanges of beams, compounds, and plate girders, whether cased or uncased, may be stressed to the same limits as tension flanges, provided that adequate lateral restraint is available. Compressive stresses should be based on the moment of inertia of the gross sectional area, but permission is given to use the nett sectional area as for the tension flange, and this is usually done to save labour in calculation and to keep both flanges the same section.

Where adequate lateral restraint is not available, permissible compressive stress in flanges is limited by the following formula :—

$$F_{bc} = \frac{1000}{L/r} \times K_1 \text{ tons per square inch}$$

where F_{bc} = the allowable compressive stress

L = maximum distance between effective lateral restraints.

r = radius of gyration perpendicular to the plane of bending (usually the minimum radius of gyration).

K_1 = a factor which is unity for unsymmetrical sections and for symmetrical sections has the values given in Table 4.

This formula is a considerable improvement on previous practice based solely on the width of the compression flange, and enables the designer to take advantage of the greater stiffness obtained by careful proportioning of the cross sectional area. It appears at first sight that much more laborious calculation is involved in the new formula, but an increasing use and understanding of the procedure will eventually dispel initial uncertainties. The main difficulty will be found in ascertaining values of the factor K_1, which is based on the ratio r_{xx}/r_{yy}. Particularly will this be the case with compound sections, and the designer would be well advised

to work out the values of this ratio for the compound sections in most general use.

TABLE 4. (See Table 10 in B.S.S. 449).

Bending Stress Factors K_1.

$\frac{r_{xx}}{r_{yy}}$	K_1	$\frac{r_{xx}}{r_{yy}}$	K_1
5·0	1·000	3·9	1·275
4·9	1·025	3·8	1·300
4·8	1·050	3·7	1·325
4·7	1·075	3·6	1·350
4·6	1·100	3·5	1·375
4·5	1·125	3·4	1·400
4·4	1·150	3·3	1·425
4·3	1·175	3·2	1·450
4·2	1·200	3·1	1·475
4·1	1·225	3·0 or less	1·500
4·0	1·250		

For cased beams, that is to say for beams cased in accordance with Clause 19d of the specification, the value of r_{yy} in certain cases may be taken as $0{\cdot}2\ (b+4)$ where b is the width of the compression flange, but in no case may the stress in a cased beam exceed the allowable stress for the same beam uncased by more than 50 per cent. This means in practice that the permissible compressive stress in cased beams, where less than 10 tons, must be checked for both the cased and the uncased condition.

Table 5 gives values of bending stress factors K_1 for British standard beams (uncased) and also maximum unrestrained lengths for each beam, based on a working stress of 10 tons per square inch. For longer unrestrained lengths permissible stress must be reduced proportionately.

An examination of this table will show that deep beams with relatively narrow flanges, such as $18'' \times 6''$, $15'' \times 5''$ and $13'' \times 5''$ can be stressed to maximum value for comparatively short unrestrained lengths only, so that when used for spans longer than the given maximum unrestrained length they may only prove economical under conditions giving adequate lateral restraint, that is, with incoming beams at short intervals.

For shear stress a general maximum permissible stress (except in the case of rivets) has been fixed at 6·5 tons per square inch for unstiffened webs (usually of beams and compounds), with the proviso that the ratio of web thickness to depth must not exceed one seventy-fifth. In the case of joists and channels the depth refers to the clear distance between flanges (ignoring fillets) and in the case of riveted plate girders to the clear distance between flange angles.

TABLE 5.

Values of Bending Stress Factor K_1.

For British Standard Beams (Uncased).

With Maximum Unrestrained Lengths (For F_b = 10 Tons/Sq. in).

Section	Weight per Foot lbs.	r_{xx}	r_{yy}	$\frac{r_{xx}}{r_{yy}}$	K_1	Maximum Unrestrained Length.
24 × 7½	95	9·52	1·50	6·4	1·000	12′-6″
22 × 7	75	8·72	1·36	6·4	1·000	11′-4″
20 × 7⅓	89	7·99	1·55	5·2	1·000	12′-11″
20 × 6½	65	8·01	1·31	6·1	1·000	10′-11″
18 × 8	80	7·41	1·72	4·3	1·175	16′-10″
18 × 7	75	7·22	1·45	5·0	1·000	12′-1″
18 × 6	55	7·21	1·21	5·9	1·000	10′-1″
16 × 8	75	6·64	1·76	3·8	1·300	19′-0″
16 × 6	62	6·31	1·22	5·2	1·000	10′-2″
16 × 6	50	6·48	1·24	5·2	1·000	10′-4″
15 × 6	45	6·10	1·23	5·0	1·000	10′-3″
15 × 5	42	5·89	0·98	6·0	1·000	8′-2″
14 × 8	70	5·85	1·80	3·3	1·425	21′-3″
14 × 6	57	5·64	1·29	4·4	1·150	12′-4″
14 × 6	46	5·71	1·26	4·5	1·125	11′-9″
13 × 5	35	5·25	1·03	5·1	1·000	8′-7″
12 × 8	65	5·05	1·85	2·7	1·500	23·1
12 × 6	54	4·86	1·33	3·7	1·325	14′-8″
12 × 6	44	4·94	1·30	3·8	1·300	14′-1″
12 × 5	32	4·84	1·01	4·8	1·050	8′-10″
10 × 8	55	4·22	1·84	2·3	1·500	23′-0″
10 × 6	40	4·17	1·36	3·1	1·475	16′-8″
10 × 5	30	4·06	1·05	3·9	1·275	11′-2″
10 × 4½	25	4·08	0·94	4·4	1·150	9′-0″
9 × 7	50	3·76	1·65	2·3	1·500	20′-7″
9 × 4	21	3·62	0·82	4·4	1·150	7′-10″
8 × 6	35	3·34	1·38	2·4	1·500	17′-3″
8 × 5	28	3·29	1·11	3·0	1·500	13′-10″
8 × 4	18	3·24	0·81	4·0	1·250	8′-5″
7 × 4	16	2·89	0·84	3·5	1·375	9′-7″

For Longer Unrestrained Lengths Reduce F_b Proportionately.

In effect this means that all joists and channels may be stressed to 6·5 tons in shear. For plate girders with 6″ × 6″ flange angles maximum shear stress of 6·5 tons may be used for $\frac{3}{8}$″ webs up to 40″ deep, for $\frac{1}{2}$″ webs up to 49″ deep, and for $\frac{5}{8}$″ webs up to 59″ deep.

For stiffened webs, that is, for plate girders with intermediate stiffeners, webs may be stressed in shear either to 6·5 tons or to the value given by the following formula, whichever is the lesser.

$$F^1_s = \left(\frac{225}{b/t}\right)^2 \left[1 + \tfrac{3}{4}\left(\frac{b}{a}\right)^2\right] \text{ tons per square inch.}$$

where a = greater unsupported dimension of web in a panel.
b = lesser unsupported dimension of web in a panel.
t = web thickness.

Dimensions a and b refer to the clear distance between the flanges and the centres of stiffeners.

Where the web thickness is less than one seventy-fifth of the unsupported depth stiffeners are to be provided for the full length of the girder at a spacing not exceeding one-and-a-half times the girder depth.

TABLE 6. (Table 11 in B.S.S. 449—Amendment No. 1).
Shear Stress in Webs for Steels to B.S. 15, 548 and 968.

d/t	Stress in tons/sq. in. Fs^1 for different ratios of depth to distance between Stiffeners.												
	1 / >1·5	1 / 1·5	1 / 1·4	1 / 1·3	1 / 1·2	1 / 1·1	1·0	1·25	1·33	1·5	1·67	1·75	2·0
75	9·00												
80							Values above Stepped Line are not applicable to Steel to B.S. 15.						
90		8·34	8·64										
100		6·75	7·00	7·31	7·70	8·20	8·86						
110		5·58	5·78	6·04	6·36	6·78	7·32						
120		4·69	4·86	5·08	5·35	5·69	6·15	8·13	8·89				
130		3·99	4·14	4·33	4·56	4·85	5·24	6·93	7·57	8·98			
140		3·44	3·57	3·73	3·93	4·18	4·52	5·97	6·53	7·75			
150		3·00	3·11	3·25	3·42	3·64	3·94	5·20	5·69	6·75	7·94	8·58	
160		2·64	2·73	2·86	3·01	3·20	3·46	4·57	5·00	5·93	6·98	7·54	
170		2·34	2·42	2·53	2·66	2·84	3·07	4·05	4·43	5·25	6·18	6·68	8·32

These Stresses apply where reductions in Web area are due only to Rivet holes, etc. Where large apertures are cut in Web special analysis is necessary to ensure that permissible stresses are not exceeded.

Table 6 gives values of permissible shear stress in webs for ratios of depth to thickness exceeding 75, but it is important to note that where these shear stresses are accompanied by bending stress in excess of 2 tons per square inch the values in Table 6 are to be reduced in accordance with Clause 22c of the specification (see Amendment No. 1 to B.S.S. 449).

In all cases except where large holes are cut in the web the permissible shear may be calculated on the gross cross section of the web, ignoring rivet holes, and in the case of joists and channels the gross section is based on the total depth of the beam.

All this is so very complicated that the designer may probably decide that in most cases it is desirable to ensure that the ratio of depth to thickness does not exceed 75. In any case a thin web reduces the bearing value of the flange rivets, and a reduction in web thickness may well be false economy.

Permissible bearing stress (except in the case of rivets) is 12 tons per square inch, calculated on the nett area in contact.

Combined Stresses.

Where members are subject to both axial compression and bending at the same time, as in the case of many stanchions, or to both axial tension and bending at the same time, procedure is very different to formerly. For combined compression and bending, members are to be so proportioned that the quantity $\frac{f_a}{F_a} + \frac{f_{bc}}{F_{bc}}$ does not exceed unity.

where f_a = actual axial compressive stress.
F_a = allowable compressive stress in axially loaded struts, as given in Table 1 in this pamphlet (Table 7 in B.S.S. 449).
f_{bc} = the sum of the actual compressive stresses due to bending, about both rectangular axes.
F_{bc} = the allowable compressive stress in bending, in accordance with Clause 19 of the specification, subject to the proviso that values of K_1 shall be taken as unity where excessive flexibility is likely to occur.

In the case of combined tension and bending, members are to be so proportioned that the quantity $\frac{f_t}{F_t} + \frac{f_{bt}}{F_{bt}}$ does not exceed unity.

where f_t = actual axial tensile stress.
F_t = allowable axial tensile stress
= 9 tons per square inch.

F_{bt} = permissible tensile stress in bending.
= 10 tons per square inch except for plate girders where F_{bt} = 9·5 tons per square inch.

From the above formulae for combined stresses it will be clear that more work is involved in designing than under the previous British Standard, and some difficulty will be experienced in the selection of trial sections, particularly in the case of stanchions subject to bending about both axes. Experience alone can resolve this difficulty, but it might be noted that where bending stresses are small compared with axial stresses, sections giving a reasonable margin of strength for axial loads alone will be found on analysis to be sufficient for combined stresses.

Deflection.

Clause 34 of the specification states that the deflection of any beam shall not exceed one three hundred and twenty-fifth of the span unless deflection is of no consequence. It states further that in any case deflection is not to be such as to impair the efficiency of the structure or cause damage to finishings. In view of the higher bending stresses now permitted it is very necessary that this point should be watched. In particular, in cases where floors support heavy machinery it may be necessary to reduce working stresses considerably in order to keep down the deflection to an acceptable figure.

Permissible stresses in rivets and bolts are given in Clause 23. Satisfactory features of this clause are the fixing of permissible stresses for bolts and rivets in tension, which previously was often a subject of dispute, and an increase of 25 per cent. in bearing values for rivets and fitted bolts in double shear, thus making provision for the additional strength provided by friction.

Stresses due to Wind Forces.

Members must first be designed, excluding wind forces, on the basis of the normal allowable stresses enumerated above, and then checked for the addition of wind stresses. Maximum stress including wind stress must not exceed the normal allowable stress without wind by more than 25 per cent.

Stresses in Grillage Beams.

Clause 27 of the specification states that stresses in grillage beams may exceed those for uncased beams by 33⅓ per cent., provided that there is a clear space of at least 3″ between the flanges, and the concrete solidly fills the spaces between the beams and gives a cover of at least 4″ all round and on top of the grillage. This permissible increase in stress is based on the assumption that

well rammed concrete and concrete encasement gives substantial assistance to the beams in resisting the load. (See also note regarding Increase in Bearing Stress in Grillage Design given in later pages).

Slab Bases for Stanchions.

Maximum permissible stress in slab bases due to loads of all kinds, including wind loads, is 12 tons per square inch. It has always been assumed that base plates could safely take a higher stress on the basis of calculation usually adopted than other members generally, and this fact is now officially recognised in the new specification.

Permissible Stresses in Welds.

These are given in Clause 24 of the specification, and constitute a much more rational procedure than was previously the case. Butt welds are to be calculated on an area equal to the effective length multiplied by the throat thickness, and permissible stresses shall not exceed the permissible tensile compressive and shear stresses for the parent metal.

Fillet welds are to be calculated on an area equal to the effective length multiplied by the throat thickness, and permissible stress must not exceed the permissible shear stress in the parent metal (normally 6·5 tons per square inch).

In the case of welds subject to the action of bending combined with direct load, the resultant stress calculated as the vector sum must not exceed the allowable stresses referred to above for butt welds and fillet welds, according to the type of weld under consideration.

This is a strictly rational procedure, based on the assumption that the weld metal and parent metal become a homogeneous mass, and must therefore be designed to the same maximum stresses.

Dynamic Loads.

A highly satisfactory feature of the new specification is the fixing of allowances for dynamic and surge effects on crane gantry girders, which was often previously a subject of controversy. Full particulars of allowances to be made are given in Clause 15 of the specification. Generally these are lower than has formerly been the case, and take a more realistic view of actual conditions. The clause also states that when vertical dynamic effect and lateral surge are considered together, gantry girders and their supports may be designed to a stress 10 per cent. in excess of normal allowable stress. When, however, wind loads are to be taken into account as well as dynamic and surge effects, it is not permissible

to increase normal allowable stress by 10 per cent. and a further 25 per cent. In such cases the maximum allowable increase in stress is 25 per cent. An important point to note in connection with crane gantry loads is that maximum wheel loads occur when the crab has reached the extreme limit of its travel. It is to this condition that dynamic and surge effects are added, and it will be appreciated that such a combination of extreme conditions will rarely occur in actual practice. It is also highly improbable that such a condition will operate at all when maximum wind pressures are to be taken into account, so that it becomes the duty of the designer to arrive at a reasonable and intelligent combination of design conditions in order to secure economy of material.

Buckling Stresses in Rolled Beam Webs.

Previous issues of the specification, while calling for beam webs to be checked against buckling, gave no guidance as to how this should be done. Clause 47a of the new specification gives detailed requirements. Permissible buckling stress in beams is to be taken from Table 1, that is, is to be the same as the permissible compressive stress for struts, and is to be based on a slenderness ratio of $\frac{d\sqrt{3}}{t}$

where d = clear depth of web, ignoring root fillets.
t = web thickness.

The assumed effective buckling area is the buckling length given in Fig. 1 multiplied by the web thickness. Providing therefore that the reaction or concentrated load does not exceed the value of $F_a \times t \times l$ obtained on the above basis, it is not necessary to stiffen the webs of rolled beams against buckling.

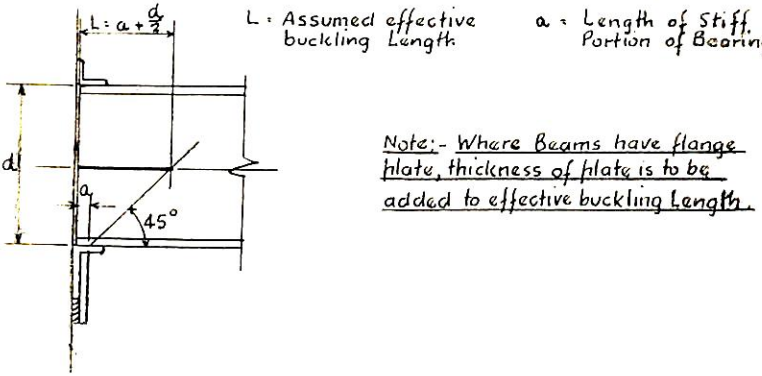
L = a + d/2
L = Assumed effective buckling Length
a = Length of Stiff Portion of Bearing
d
a
45°
Note:- Where Beams have flange plate, thickness of plate is to be added to effective buckling Length.

Fig. 1—Effective Buckling Length for Rolled Beams.

Wind Pressure.

The 1948 edition of the specification makes radical changes in the procedure to be adopted in dealing with wind pressure on a building. Clause 11 gives particulars of the method to be adopted for finding the unit wind pressure in any particular case, and Clause 12 indicates the manner in which this pressure is to be applied to the building.

Values of wind velocities to be used in design for structures in the British Isles are listed in Table 7 below :—

TABLE 7. (Table 3 in B.S.S. 449).

Wind Velocities (v) in m.p.h. for use in Design.

District.	Where Natural Protection is afforded.	Open Country Inland.	Condition of Maximum Exposure.
West Scotland N.W. Ireland N.W. England Wales S.W. England	60	70	80
East Scotland N.E. Ireland N.E. England East England South and S.E. England	55	65	75
Central England (including Severn Estuary)	50	60	70

Unit wind pressure (p) in lbs. per square foot applied uniformly over the vertical faces of a building are obtained from the following formula :—

$$p = \frac{v^2}{600} \sqrt{1 + 0{\cdot}06\ (h - s)}$$

where v = wind velocity in m.p.h. at height of 40′-0″ above ground on an open site.

h = height in feet of the windward vertical face above general ground level.

s = height in feet above general ground level assumed sheltered by permanent obstacles.
(value of s not to exceed $\frac{1}{2}h$, and value of $h - s$ not to be taken as less than 10′-0″).

Table 8. (Table 4 in B.S.S. 449).

Unit Wind Pressure (p) lbs. per Sq. Ft.

Effective Height $(h-s)$	Wind Velocity (v) in miles per hour.						
	50	55	60	65	70	75	80
10	5·3	6·4	7·6	8·9	10·3	11·9	13·5
15	5·7	6·9	8·2	9·7	11·2	12·9	14·7
20	6·2	7·5	8·9	10·4	12·1	13·9	15·8
25	6·6	8·0	9·5	11·2	12·9	14·9	16·9
30	7·0	8·4	10·0	11·8	13·7	15·7	17·9
35	7·4	8·9	10·6	12·4	14·4	16·5	18·8
40	7·7	9·3	11·1	13·0	15·0	17·2	19·6
45	8·0	9·7	11·6	13·5	15·7	18·1	20·5
50	8·3	10·1	12·0	14·1	16·3	18·7	21·3
60	9·0	10·8	12·9	15·1	17·3	20·1	22·9
70	9·5	11·5	13·7	16·1	18·4	21·4	24·3
80	10·0	12·1	14·4	16·9	19·6	22·5	25·6
90	10·5	12·7	15·1	17·7	20·6	23·6	26·9
100	11·0	13·3	15·8	18·6	21·6	24·8	28·2
120	11·9	14·4	17·1	20·1	23·4	26·8	30·5
140	12·8	15·4	18·4	21·6	25·0	28·7	32·7
160	13·6	16·4	19·6	22·9	26·6	30·5	34·8
180	14·3	17·3	20·6	24·2	28·0	32·1	36·6
200 and over	15·0	18·2	21·6	25·4	29·4	33·8	38·4

For Heights above 200′-0″ no increase of Pressure need be allowed.

Table 8 gives values of unit wind pressure (p) for various heights and wind velocities, based on the above formula.

Clause 12 of the specification gives detailed instructions concerning the manner in which wind pressure is to be applied to a building. For external wind pressure on the vertical sides half of the unit pressure (p) is to be assumed acting on the windward side, and half as a suction effect on the leeward side. For a flat roof an upward suction effect equal to (p) is to be assumed acting on the windward half of the roof, and an upward suction effect of $\frac{1}{2}p$ on the leeward half. For a pitched roof, irrespective of the slope, a suction effect of $\frac{1}{2}p$ is to be assumed on the leeward slope, acting normal to the surface. On the windward slope, with a slope of 30 degrees, no wind pressure is to be taken. For a slope of $22\frac{1}{2}$ degrees, a negative pressure (that is, a suction effect) of $\frac{1}{4}p$ is to be assumed. For a slope of 45 degrees, a positive pressure of $\frac{1}{4}p$ is to be taken, and for slopes of 70 degrees and over a positive pressure of $\frac{1}{2}p$ is to be assumed.

It will be noted that these values are very different from earlier practice, and involve a radical change in procedure for roof truss design.

For curved roofs wind pressures are to be the same as for a pitched roof of the same rise, but are to be assumed to act radially inwards and outwards as the case may be.

For multi-span roofs, provision is made in the specification for reductions in wind pressures on all spans except the windward span.

It is also necessary to make provision for internal air pressure and suction in suitable cases. Where walls are mostly bricked there will be no appreciable internal pressure, but in the case of sheeted buildings where air can seep through the joints and ventilators an allowance must be included. In the nature of the case it is difficult to assess this effect with any exactness, but the specification calls for an allowance of $0{\cdot}2\,p$ internal pressure or alternatively $0{\cdot}2\,p$ internal suction, acting normal to the walls and roof slope, and this should cover most conditions. This effect is to be allowed in addition to the external pressures and suctions previously mentioned.

Where buildings have large openings, such as hangars, the above allowances of $0{\cdot}2\,p$ pressure or suction are to be increased to $0{\cdot}5\,p$.

The whole of the preceding allowances for wind are intended as average pressures applicable to the building as a whole. In addition provision must be made for the effect of local gusts, which will effect such items as sheeting rails and purlins, and also roof trusses on small spans. To allow for this local effect the specification calls for an addition of $0{\cdot}1\,p$ to the pressure and suction.

From this discussion it will be appreciated that wind analysis is much more complicated than previously, but the designer will find in many cases that it is possible to lump all these effects together without doing violence to the specification. In the case of roof trusses of average span and usual rise, it will also be found that so long as all members are of strut section wind effects can be ignored altogether and designs prepared solely for dead load and snow load, which appears to be getting back to the procedure adopted in earlier times. It does not of course follow from this that wind effects can also be ignored in designing supports for roof trusses. Indeed, the reverse is the case, and a full analysis must be made in order to ascertain the effects of wind pressures and wind suctions on such members.

PART 2.

DETAIL DESIGN.

Roof Trusses 55′-0″ span, 15′-0″ centres, with Rise of 1 in 2.

Height from ground to eaves = 30′-0″.

Covering—Big Six asbestos cement corrugated sheets with at least one-third of roof area covered with patent glazing.

Line diagram of half truss, showing sheets, glazing, and purlins is given in Fig. 2.

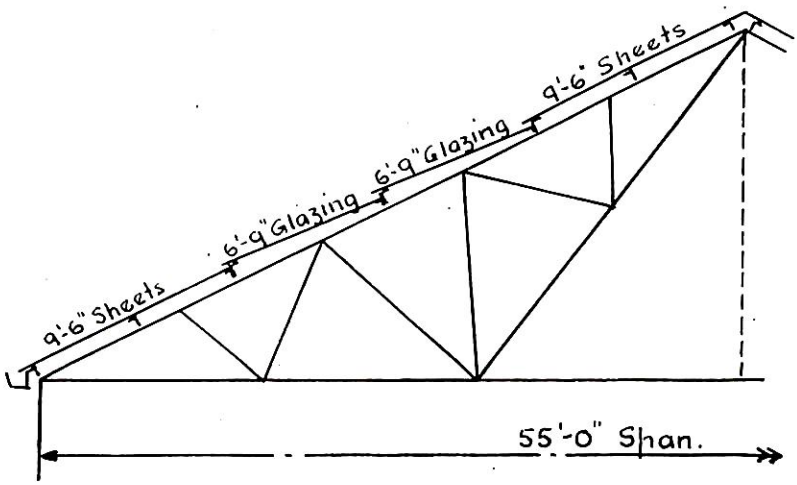

Fig. 2—Layout of Truss and Covering.

Snow load = 10 lbs. per square foot of horizontal projection

$$= \frac{10 \times 55 \times 15}{2240} = 3{\cdot}7 \text{ tons per truss.}$$

Dead load = 10 lbs. per square foot of horizontal projection (assumed).

$$= \frac{10 \times 55 \times 15}{2240} = 3{\cdot}7 \text{ tons per truss.}$$

Total vertical load per truss = 7·4 tons (say 7½ tons).

Vertical load per panel of rafters = 0·75 ton.

Wind Load.

Assumed wind velocity = 60 miles per hour.

Unit wind pressure (p) for height of 30′-0″ = 10 lbs. per square foot.

For rise of 1 in 2, wind pressure on windward slope = $-0{\cdot}125\,p$
(*i.e.*, a suction effect of $0{\cdot}125\,p$).
Wind pressure on leeward slope = $-0{\cdot}5\,p$.
Internal wind pressure on each slope = $\pm\,0{\cdot}2\,p$.
Resultant wind pressure—windward slope
= $-0{\cdot}325\,p$ or $+\,0{\cdot}075\,p$
= $-$ 3·25 lbs. per square foot or $+$ 0·75 lb. per square foot.
Resultant wind pressure—leeward slope = $-\,0{\cdot}7\,p$.
= $-$ 7 lbs. per square foot.

Rafter length (for rise of 1 in 2) = $27{\cdot}5 \times 1{\cdot}118 = 30{\cdot}75$ ft.
(For construction say 30′-10″).

Maximum positive wind pressure—windward slope

$$= \frac{0{\cdot}75 \times 30{\cdot}8 \times 15}{2240} = +\ 0{\cdot}15 \text{ ton.}$$

Maximum negative wind pressure—windward slope

$$= \frac{3{\cdot}25 \times 30{\cdot}8 \times 15}{2240} = -\ 0{\cdot}7 \text{ ton.}$$

Maximum negative wind pressure—leeward slope

$$= \frac{7 \times 30{\cdot}8 \times 15}{2240} = -\ 1{\cdot}5 \text{ tons.}$$

Gust effect (assumed acting only on purlins)
= $\pm\ 0{\cdot}1p$ = $\pm$ 1 lb. per sq. ft.

Centre Sheeting Purlins—4′-6″ spacing.

Snow load	=	10 lbs.
Sheets	=	3·5 lbs.
Own weight	=	1·5 lbs.
		15 lbs. per square foot.

$$\text{Vertical load per purlin} = \frac{15 \times 15 \times 4{\cdot}5}{2240} = 0{\cdot}45 \text{ ton.}$$

$$\text{Normal component} = \frac{0{\cdot}45}{1{\cdot}118} = 0{\cdot}4 \text{ ton.}$$

Maximum wind pressure (including gust effect)
= 1·75 lbs. per square foot.

This is less than 25 per cent. of snow load + dead load, and wind effect may therefore be ignored. (See also Clause 55 of specification for confirmation).

Size of purlin and required section modulus are also conditioned by requirements of Clause 55.

Minimum depth of purlin $= \frac{L}{45} = \frac{15 \times 12}{45} = 4''$

Minimum width of purlin $= \frac{L}{60} = \frac{15 \times 12}{60} = 3''$

Minimum Z to be provided $= \frac{W\,L}{90} = \frac{0{\cdot}4 \times 15 \times 12}{90} = 0{\cdot}8$ ins.^{3}

(For a stress of 10 tons per square inch the above section modulus is based on a bending moment of WL/9, or alternatively on a bending moment of WL/10 for a stress of 9 tons per square inch. This is not out of the way for the construction usually adopted (particularly as the snow load represents a fall nearly 2'-0" deep).

Use $4 \times 3 \times \frac{5}{16}$ Angle. Z given = 1·2 ins.^{3}.

It may be noted that the centres of trusses would have to be reduced to 12'-6" to enable $3\frac{1}{2}'' \times 2\frac{1}{2}''$ angles to be used.

Eaves and Apex Purlins.

For uniformity **use $4 \times 3 \times \frac{5}{16}$ angle** as minimum permissible. Load on the apex purlins will only be about half of that on the centre sheeting purlins, but specification requirements prevent a reduction in section. Load on the eaves purlins will be very nearly equal to that on the centre sheeting purlins, as although roof load is halved the eaves purlin usually supports the gutter and water in same.

Centre Glazing Purlins—6'-6" spacing.

Snow load	=	10 lbs.
Glazing	=	6 lbs.
Own weight	=	2 lbs.
		18 lbs. per square foot.

Vertical load per purlin $= \frac{18 \times 15 \times 6{\cdot}5}{2240} = 0{\cdot}78$ ton.

Normal component $= \frac{0{\cdot}78}{1{\cdot}118} = 0{\cdot}7$ ton.

Minimum Z required $= \frac{W\,L}{90} = \frac{0{\cdot}7 \times 15 \times 12}{90} = 1{\cdot}4$ ins.^{3}

Use $5 \times 3 \times \frac{5}{16}$ Angle—Z given = 1·84 ins.^{3}

Top and Bottom Glazing Purlins.

For uniformity **use $5 \times 3 \times \frac{5}{16}$ Angle.**

TABLE 9.

Loads in Roof Truss Members.

Member	D.L.	S.L.	W.L.	D.L.+S.L.	D.L.+W.L.	D.L.+S.L. +W.L.
B-1	−3·8	−3·8	+1·8	−7·6	−2·0	−5·8
C-2	−3·5	−3·5	−1·7	−7·0	−1·8	−5·3
D-4	−2·9	−2·9	+1·5	−5·8	−1·4	−4·3
E-6	−3·1	−3·1	+1·7	−6·2	−1·4	−4·5
F-7	−3·1	−3·1	+1·8	−6·2	−1·3	−4·4
1-2	−0·4	−0·4	+0·2	−0·8	−0·2	−0·6
2-3	+0·3	+0·3	−0·1	+0·6	+0·2	+0·5
3-4	−0·5	−0·5	+0·2	−1·0	−0·3	−0·8
4-5	−0·5	−0·5	+0·2	−1·0	−0·3	−0·8
5-6	+0·3	+0·3	−0·1	+0·6	+0·2	+0·5
6-7	−0·4	−0·4	+0·2	−0·8	−0·2	−0·6
7-8	+1·5	+1·5	−0·6	+3·0	+0·9	+2·4
5-8	+1·1	+1·1	−0·5	+2·2	+0·6	+1·7
N-1	+3·4	+3·4	−1·4	+6·8	+2·0	+5·4
N-3	+3·0	+3·0	−1·2	+6·0	+1·8	+4·8
N-8 N-9	+1·9	+1·9	−0·7	+3·8	+1·2	+3·1
8-9	0	0	0	0	0	0
N-14	+3·0	+3·0	−1·8	+6·0	+1·2	+4·2
N-16	+3·4	+3·4	−2·1	+6·8	+1·3	+4·7
9-12	+1·1	+1·1	−1·0	+2·2	+0·1	+1·2
9-10	+1·5	+1·5	−1·3	+3·0	+0·2	+1·7
10-11	−0·4	−0·4	+0·3	−0·8	−0·1	−0·5
11-12	+0·3	+0·3	−0·2	+0·6	+0·1	+0·4
12-13	−0·5	−0·5	+0·5	−1·0	0	−0·5
13-14	−0·5	−0·5	+0·5	−1·0	0	−0·5
14-15	+0·3	+0·3	−0·2	+0·6	+0·1	+0·4
15-16	−0·4	−0·4	+0·3	−0·8	−0·1	−0·5
H-10	−3·1	−3·1	+2·2	−6·2	−0·9	−4·0
J-11	−3·1	−3·1	+2·1	−6·2	−0·9	−4·0
K-13	−2·9	−2·9	+1·7	−5·8	−1·2	−4·1
L-15	−3·5	−3·5	+2·1	−7·0	−1·4	−4·9
M-16	−3·8	−3·8	+2·2	−7·6	−1·6	−5·4

D.L. = Dead Load. S.L. = Snow Load. W.L. = Wind Load. − indicates Compression. + indicates Tension. All Loads in tons.

Table 9 gives loads in roof truss members. These have been listed in more detail than is actually necessary, in order to demonstrate the important fact that for a rise of 1 in 2, which is the normal proportion adopted for factory roofs with this type of covering, wind forces need not be taken into consideration at all, the worst condition for all members being the summation of dead load and snow load. This is a most important result of the revised wind allowances given in the new specification, and greatly simplifies procedure. It does not necessarily follow that wind can be neglected irrespective of the span, and still less if the trusses have a greater rise than in this case, but it is clear from this detailed

table that for the most usual spans adopted in factory construction, with a rise of 1 in 2, wind can be neglected so far as the truss design is concerned. The wind in fact reduces the effect of dead load and snow load.

Reciprocal diagrams for the roof truss are not given here, as these follow normal procedure and present no difficulty, the only point needing mention being that the diagram for wind has been drawn assuming that the horizontal component of the wind is shared equally between the two reactions, which appears to be a reasonable assumption for trusses of average span subject to the type of wind load now assumed.

Tension Members.

As the effect of wind does not produce a reversal of stress in any member, it is not necessary to take into account the maximum slenderness ratio referred to in Clause 36 (4) of the specification. Nevertheless to ensure as much rigidity as possible in handling and in transport it is desirable to use angle sections throughout. Clause 39 (*b*) requires that the assumed effective area of angles in tension shall be that of the nett area of the connected leg plus half the area of the outstanding leg. This proviso is due to the fact that the connected leg will tend to take more of the load than the other leg—particularly at the connections where the nett area is least—and it seems likely that at the critical section the extreme edge of the outstanding leg is hardly stressed at all. It is assumed in the following calculations that all holes are $\frac{13}{16}$″ diameter.

End Main Tie. Members N-1 and N-3.

Maximum load = 6·8 tons tension.

Nett area required $= \frac{6{\cdot}8}{9} = 0{\cdot}75$ sq. ins.

For $\frac{1}{4}$″ metal addition for hole $= \frac{13}{16} \times \frac{1}{4} = 0{\cdot}20$ sq. ins.

For $2\frac{1}{2}$″ outstanding leg addition for half area

$$= \frac{2\frac{1}{4} \times \frac{1}{4}}{2} = 0{\cdot}28 \text{ sq. ins.}$$

Gross area required $= 0{\cdot}75 + 0{\cdot}20 + 0{\cdot}28 = 1{\cdot}23$ sq. ins.

Use 3″ × 2½″ × ¼″ Angle—area given = 1·31 sq. ins.

Centre Main Tie. Member N-8.

Load = 3·8 tons.

Nett area required $= \frac{3{\cdot}8}{9} = 0{\cdot}42$ sq. ins.

Add for hole and outstanding leg (as before) 0·48 sq. ins.
Gross area required = 0·42 + 0·48 = 0·90 sq. ins.
For construction **use $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ angle**—area given = 1·19 sq. ins.

Apex Ties. Members 7-8 and 5-8.

Maximum load = 3·0 tons.

Nett area required = $\frac{3 \cdot 0}{9}$ = 0·33 sq. ins.

For construction **use $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ angle.**

Secondary Ties. Members 2-3 and 5-6.

Maximum load = 0·6 ton.
For construction **use $2\frac{1}{2}'' \times 2'' \times \frac{1}{4}''$ angle.**

Compression Members.

Clause 36a of the specification fixes the maximum slenderness ratio, when wind is not a controlling factor, at 180, and it will often happen in roof truss and light frame design that this requirement will determine the minimum section.

Primary Struts. Members 3-4 and 4-5.

Maximum load = 1·0 ton. Length = 8′-6″ = 102″.

Minimum radius of gyration = $\frac{102}{180}$ = 0·56″.

Try $3'' \times 3'' \times \frac{1}{4}''$ angle as minimum. r = 0·59″.

$L/r = \frac{102}{0 \cdot 59} = 173.$

From Table 2, permissible stress F_c2 = 1·57 tons per sq. in.
Area of section = 1·44 sq. ins.
Permissible load = 1·44 × 1·57 = 2·3 tons.
The section is over-strong but cannot be reduced owing to slenderness ratio requirements.

Secondary Struts. Members 1-2 and 6-7.

Maximum load = 0·8 tons. Length = 4′-3″ = 51″.

Minimum radius of gyration = $\frac{51}{180}$ = 0·28″.

Use $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ angle. Radius of gyration = 0·49″.

It is not necessary to check this section in detail as it is obviously strong enough.

Rafters. Members B-1, C-2, D-4, E-6, F-7.

The rafter is continuous with a varying compressive load in each panel. Owing to the purlins being located away from the panel points there will also be a bending moment in each panel. This condition is not specifically covered by the specification, but sufficient guidance is given to decide on an acceptable procedure.

Appendix D of the specification, in discussing permissible working stresses in discontinuous struts, assumes an effective length equal to 0·8 of the distance between the fastenings.

Appendix E of the specification, referring to effective lengths of continuous stanchions, gives various length factors for the fixity conditions noted.

Taking all this information together, it is not unreasonable to assume a length factor of 0·75 for each panel of the rafter when considering the effective length in the plane of the truss. In the other plane, however, the rafters are restrained only by the purlins. The glazing purlins are 6′-6″ centres and this figure will be assumed to be the effective length in this plane.

With the sheeting purlins spaced at 4′-6″ centres and the centre glazing purlin occurring at the centre of the middle panel, it will be assumed that the middle panel of the rafter, that is, member D-4 gives worst condition for design.

Point load from purlin = 0·78 tons.

Assumed effective length in plane of truss $= 0{\cdot}75 \times 74 = 55{\cdot}5''$.

M at centre due to purlin load $= \dfrac{0{\cdot}78 \times 55{\cdot}5}{4} = 11$ tons ins.

Direct compressive load $= 5{\cdot}8$ tons.

Try 2 angles $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ with $\frac{5}{16}''$ space between (Fig. 3a).

Area of section = 2·88 sq. ins.

$I_{xx} = 3{\cdot}5$ ins.^{4}. $I_{yy} = 2\{0{\cdot}74 + (1{\cdot}44 \times 0{\cdot}76^2)\} = 3{\cdot}14$ ins.^{4}.

$r_{xx} = \sqrt{\dfrac{3{\cdot}5}{2{\cdot}88}} = 1{\cdot}1''$ $\quad r_{yy} = \sqrt{\dfrac{3{\cdot}14}{2{\cdot}88}} = 1{\cdot}04''$.

$L_{xx} = 55{\cdot}5''$. $\quad L_{yy} = 78''$.

L/r (Axis $_{xx}$) $= \dfrac{55{\cdot}5}{1{\cdot}1} = 50{\cdot}5$ $\quad L/r$ (Axis $_{yy}$) $= \dfrac{78}{1{\cdot}04} = 75$.

F_a (Table 1) = 5·36 tons per sq. in.

$F_{bc} = \dfrac{1000}{L/r} = \dfrac{1000}{75} = > 10$ tons (maximum) per sq. in.

$f_a = \dfrac{5{\cdot}8}{2{\cdot}88} = 2{\cdot}04$ tons per sq. in.

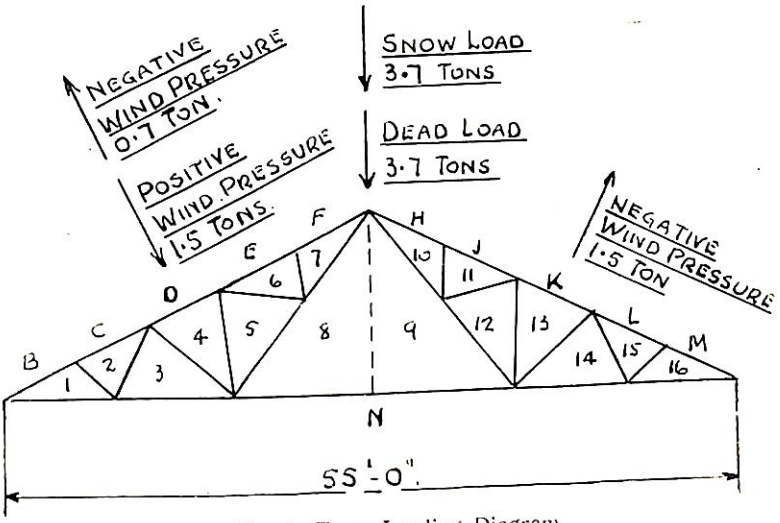

Fig. 3—Truss Loading Diagram.

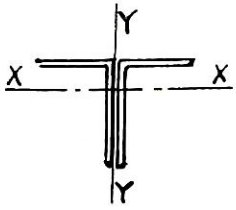

Fig. 3a.

$$Z_{xxc} = \frac{3{\cdot}5}{1{\cdot}09} = 3{\cdot}2 \text{ ins.}^3 \quad f_{bc} = \frac{11}{3{\cdot}2} = 3{\cdot}44 \text{ tons per sq. in.}$$

$$\frac{f_a}{F_a} = \frac{2{\cdot}04}{5{\cdot}36} = 0{\cdot}38 \quad \frac{f_{bc}}{F_{bc}} = \frac{3{\cdot}44}{10} = 0{\cdot}34$$

$$\frac{f_a}{F_a} + \frac{f_{bc}}{F_{bc}} = 0{\cdot}38 + 0{\cdot}34 = 0{\cdot}72.$$

The factor 0·72 is less than unity and the section is therefore strong enough from the point of view of compression in the upper fibres.

It is necesssary also to check the tension in the lower fibres.

$$I_{xx} = 3{\cdot}5. \quad Z_{xxt} = \frac{3{\cdot}5}{2{\cdot}41} = 1{\cdot}45 \text{ ins.}^3.$$

$$f_{bt} = \frac{11}{1{\cdot}45} = 7{\cdot}59 \text{ tons per sq. in.}$$

From direct compression $f_a = 2{\cdot}04$ tons per sq. in.
$\therefore$ Nett tensile stress in lower fibre $= 7{\cdot}59 - 2{\cdot}04$
$= 5{\cdot}55$ tons per sq. in.

The section is therefore satisfactory.

Use 2 Angles $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$.

General Notes.

(1) The truss loads are based on the trusses being spaced at 15′-0″ centres, which is wider than standard English practice in the past. Even so it will be seen that some of the members are too strong for the loads, and would not be increased if the spacing of the trusses were even wider. Taking into consideration the purlin sections it may be assumed for normal roof truss construction that a spacing of not less than 15′-0″ is desirable for economy of construction.

(2) The procedure for rafter design has been based on a triangulation which gives bending in the rafter due to purlins being located off the truss panel points. The only way to avoid this with sheeting purlins spaced at 4′-6″ centres is to include additional web members and to make the rafter panels of unequal lengths. If this is done, the rafter may then be designed purely as a compression member and will accordingly be a lighter section. Against this there will be more weight and more workmanship in the web members, and in each case the designer must balance the loss against the gain in order to arrive at the most economical layout. As many cases will occur in which bending has to be provided for, it has been thought advisable to take such a case as the basis for design.

(3) Attempts have been made to design truss rafters, subject to bending, as continuous beams with fixed ends. Unfortunately the intermediate supports are not simple supports in the sense understood with continuous beams, but have a certain measure of fixity, and the procedure cannot therefore give anything more than approximate results. The method is also rather lengthy and involved, and the time spent in such a detailed analysis does not appear to be justified in the particular conditions obtaining.

Crane Gantry Girders—30′-0″ Span.

Specification.

Load = Two 10-ton electric overhead travelling cranes.
Weight of crab = 2 tons.

Static wheel load (2 per end carriage) = 10 tons.
End carriage wheel centres = 10′-0″.
Impact addition to vertical loads = 25% = 2.5 tons.
Surge effect per crane = 10% of (10 + 2) = 1·2 tons.
= 0·3 ton per wheel.

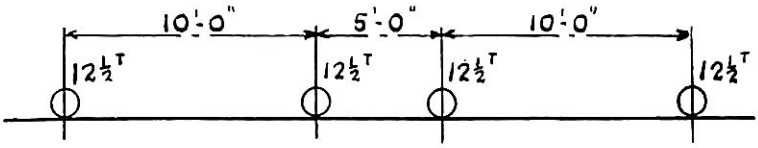

Fig. 4—Loading Diagram.

Loading diagram giving worst condition of loading is shown in Fig. 4, with the two cranes working close together. This condition will usually be realised when both cranes are used for lifting a single load.

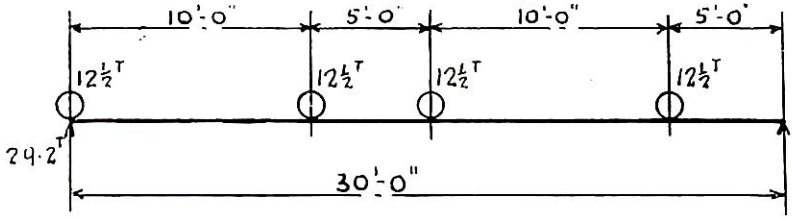

Fig. 5—Assumed Condition for Max. Shear.

Maximum reaction, and maximum surge effect on the reaction will occur with the loads in the position shown in Fig. 5.

$$\text{Maximum reaction for loads} = \frac{12\frac{1}{2}}{30}(5+15+20+30)$$

$$= \frac{12\frac{1}{2} \times 70}{30} = 29{\cdot}2 \text{ tons.}$$

$$\text{Maximum surge effect at reaction} = 29{\cdot}2 \times \frac{0{\cdot}3}{12{\cdot}5} = 0{\cdot}7 \text{ ton.}$$

The above values are of course for a single span, and the maximum reaction so found is generally used for designing the girder web, although the actual maximum shear in the girder will occur when the wheels are moved very slightly to the right of the position shown, and will therefore be slightly less than 29·2 tons.

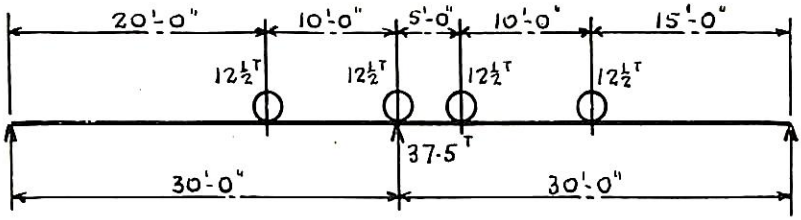

Fig. 6—Condition for Max. Reaction.

For the purpose of designing the supports, the maximum reaction will occur with wheels in the position shown in Fig. 6.

$$\text{Maximum reaction} = \frac{12\frac{1}{2}}{30}(15+25+20+30)$$

$$= \frac{12\frac{1}{2} \times 90}{30} = 37{\cdot}5 \text{ tons.}$$

$$\text{Maximum surge effect at reaction} = \frac{37{\cdot}5 \times 0{\cdot}3}{12{\cdot}5} = 0{\cdot}9 \text{ ton.}$$

Maximum bending moment may occur with either three or four wheels on the span, and both cases must be tried.

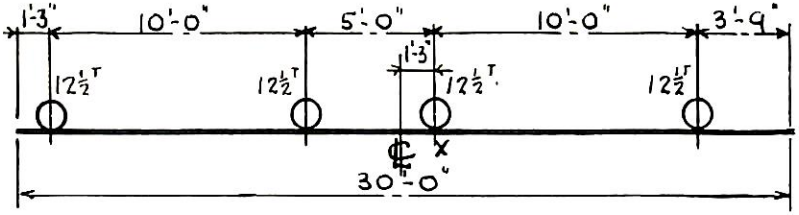

Fig. 7—Condition for Max. B.M. 4 Wheels on Span.

Case 1—Four Wheels on Span—Fig. 7.

Maximum bending moment occurs under wheel x when this wheel is as far to the right of the centre line of the span as the centre of gravity of the load system is to the left of the centre line.

B.M. for loads under wheel x

$$= \frac{12\frac{1}{2}}{30}\{13{\cdot}75\,(1{\cdot}25+11{\cdot}25+16{\cdot}25)+(3{\cdot}75\times16{\cdot}25)\}\,12$$

$$= \frac{12\frac{1}{2}}{30}\{(13{\cdot}75\times28{\cdot}75)+(3{\cdot}75\times16{\cdot}25)\}\,12$$

$$= 2281 \text{ tons ins.}$$

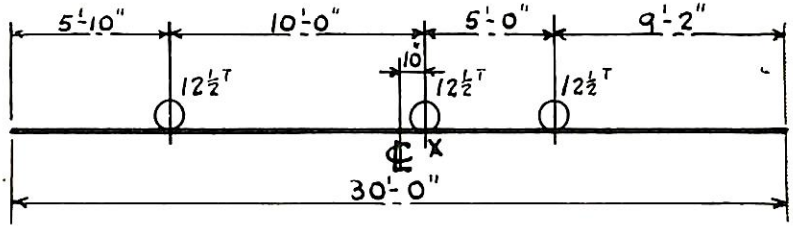

Fig. 8—Condition for Max. B.M. 3 Wheels on Span.

Case 2—Three Wheels on Span—Fig. 8.

As in the previous case maximum bending moment occurs under wheel x when this wheel is as far to the right of the center line of the span as the centre of gravity of the load system is to the left of the centre line.

B.M. for loads under wheel x

$$= \frac{12\frac{1}{2}}{30} \{ 14{\cdot}17 \ (5{\cdot}83 + 15{\cdot}83) + (15{\cdot}83 \times 9{\cdot}17) \} 12$$

$$= \frac{12\frac{1}{2}}{30} \{ (14{\cdot}17 \times 21{\cdot}66) + (15{\cdot}83 \times 9{\cdot}17) \} 12$$

$$= 2260 \text{ tons. ins.}$$

Maximum moment therefore occurs with four wheels on the span.

Allowance for Weight of Girder.

Assume weight of $1\frac{1}{2}$ cwts. per lineal foot = 2·25 tons per girder.
Additional reaction for single span = 1·13 tons.
Total reaction for single span = 29·2 + 1·13 = 30·33 tons.
Additional reaction for contiguous spans = 2·25 tons.
Total reaction for contiguous spans = 37·5 + 2·25 = 39·75 tons.

Additional B.M. under wheel x in Fig. 7 $= \frac{1\frac{1}{2}}{20} \times \frac{16{\cdot}25 \times 13{\cdot}75}{2} \times 12$

$= 101$ tons ins.

Total B.M. for design = 2281 + 101 = 2382 tons ins.

Trial Section.

In order to minimise deflection, which should be less in the case of a crane gantry than for ordinary floor beams taking static loads, it is usual to make the depth of the girder about 1/15th of the span for loads of this nature. This automatically fixes the basis joist (for trial purposes) at $24'' \times 7\frac{1}{2}''$. Lateral surge is most conveniently

taken by adding a channel to the top flange, and until the specification was revised crane gantry girders were usually constructed in this manner, with flats riveted if necessary to the bottom flange to give the required strength. Under the new procedure it becomes desirable to keep the moment of inertia and radius of gyration about the minor axis as high as possible, and this can most conveniently be done by introducing a channel on the bottom flange of the same section as the top flange, which also has the advantage of maintaining a symmetrical section about both axes, thus taking advantage of the factor K_1 in calculating permissible bending stress.

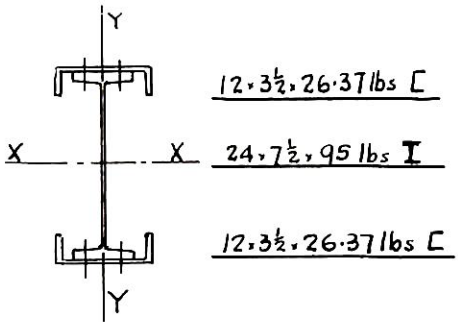

Fig. 9—Trial Section.

The trial section is shown in Fig. 9, and consists of a $24'' \times 7\frac{1}{2}'' \times 95$ lbs. R.S.J. with a $12'' \times 3\frac{1}{2}''$ channel on each flange.

Gross $I_{xx} = 2533 + 2\{ 7{\cdot}15 + (7{\cdot}76 \times 11\frac{1}{2}^2) \} = 2533 + 2067$
$= 4600$ ins.^{4}.

Gross Area of Section = 43·46 sq. ins.

$$r_{xx} = \sqrt{\frac{4600}{43{\cdot}46}} = 10{\cdot}3''.$$

Gross $I_{yy} = 62{\cdot}54 + (2 \times 159{\cdot}73) = 382$ ins.^{4}.

$$r_{yy} = \sqrt{\frac{382}{43{\cdot}46}} = 2{\cdot}97''.$$

$$\frac{r_{xx}}{r_{yy}} = \frac{10{\cdot}3}{2{\cdot}97} = 3{\cdot}5. \quad \therefore K_1 = 1{\cdot}375 \text{ (Table 4)}.$$

Assuming girder has lateral restraint at the supports only, then

$$F_{bc} \text{ (for vertical loads)} = \frac{1000}{l/r} \times K_1 = \frac{1000 \times 2{\cdot}97}{30 \times 12} \times 1{\cdot}375$$
$= >10$ tons (maximum) per sq. inch.

$\therefore$ F_{bc} for vertical loads and surge effect $= 10 + 10\%$
$= 11$ tons per sq. inch

Z_{xx} required for vertical loads only $= \frac{2382}{10} = 238{\cdot}2$ ins.^{3}.

Gross Z_{xx} given $= \frac{4600}{12{\cdot}38} = 372$ ins.^{3}.

$f_{cxx} = \frac{2382}{372} = 6{\cdot}4$ tons per square inch.

Lateral surge moment $= 2281 \times \frac{0{\cdot}3}{12{\cdot}5} = 54{\cdot}7$ tons ins.

Gross Z_{yy} given $= \frac{382}{6} = 63{\cdot}7$ ins.^{3}.

$f_{cyy} = \frac{54{\cdot}7}{63{\cdot}7} = 0{\cdot}86$ ton per square inch.

Total $f_c = 6{\cdot}4 + 0{\cdot}86 = 7{\cdot}26$ tons per square inch.

If it is assumed that all the surge effect is taken by the top half only of the girder, as is sometimes done, then the stress about axis YY is doubled, and the total combined stress becomes 8·12 tons.

Normally it is necessary to check the strength of the tension flange after allowing for holes, although where as in this case the margin of strength is ample this is not necessary. The procedure for the tension flange is as follows :—

Gross $I_{xx} = 4600$ ins.^{4}.
Allowing one hole $\frac{15}{16}$" diameter deducted for each flange.
Deduction for holes $= 2 \times \frac{15}{16} \times 1{\cdot}39 \times 11{\cdot}68^2 = 355$ ins.^{4}.
Nett $I_{xx} = 4600 - 355 = 4245$ ins.^{4}.

Nett $Z_{xx} = \frac{4245}{12{\cdot}38} = 343$ ins.^{3}.

$f_{txx} = \frac{2382}{343} = 7$ tons per square inch.

If the bottom flange is assumed to take half of the surge effect, the value of f_{tyy} must be added to f_{txx}.

Gross $I_{yy} = 382$ ins.^{4}.
Deduction for holes $= 2 \times \frac{15}{16} \times 1{\cdot}39 \times 2\frac{1}{4}^2 = 26$ ins.^{4}.
Nett $I_{yy} = 382 - 26 = 356$ ins.^{4}.

Nett $Z_{yy} = \frac{356}{6} = 59{\cdot}3$ ins.^{3}.

$f_{tyy} = \frac{54{\cdot}7}{59{\cdot}3} = 0{\cdot}92$ tons per square inch.

Total $f_t = 7 + 0{\cdot}92 = 7{\cdot}92$ tons per square inch.

Actual stresses are well below permissible, but it is undesirable to reduce the depth, owing to deflection requirements. Neither is it desirable to reduce the channel section in the flanges as this would reduce the values of Z_{yy} and r_{yy}, and thus reduce the permissible working stress as well as increasing the actual stress.

Deflection.

It will be sufficiently accurate for this purpose to reduce the loads to an equivalent uniformly distributed load.

Total B.M. under wheel x. Fig. 7 = 2382 tons ins.

E.U.D.L. per lineal foot $= \frac{2382 \times 2}{12 \times 16{\cdot}25 \times 13{\cdot}75} = 1{\cdot}777$ tons.

E.U.D.L. on 30′-0″ span $= 1{\cdot}777 \times 30 = 53{\cdot}31$ tons.

Deflection $= \frac{5}{384} \times \frac{53{\cdot}31 \times 30^3 \times 12^3}{13000 \times 4245} = 0{\cdot}58'' = \frac{1}{620}$ of span.

This is not excessive but is sufficient for a crane gantry, and explains the $\frac{\text{depth}}{\text{span}}$ ratio adopted.

Rivet pitch in flanges—Assumed maximum shear = 30·33 tons.

Horizontal shear per foot at ends $= \frac{12\ VG}{I}$

where V = Vertical shear.
G = First moment of flange channel area about neutral axis of girder.
I = Moment of Inertia of complete section.

Horizontal shear per foot at ends

$= \frac{12 \times 30{\cdot}33 \times 7{\cdot}76 \times 11{\cdot}5}{4245} = 7{\cdot}7$ tons.

No. of rivets required per foot in each flange $= \frac{7{\cdot}17}{4{\cdot}14} = 2$

Specification requires pitch in compression flange to be not more than 6″ or $16 \times t$, whichever is the lesser, value of t in this case being the thickness of the channel web. For uniformity rivet pitch in both flanges will be made 6″ on line and staggered.

Web Buckling.

Section must be checked for buckling in accordance with procedure given in Clause 47a of the specification.

Depth of web—d—ignoring root fillets = 21·41″.
Web thickness—t = 0·57″.

$$\text{Ratio of slenderness} = \frac{d\sqrt{3}}{t} = \frac{21{\cdot}41 \times 1{\cdot}73}{0{\cdot}57} = 65.$$

Permissible stress (Table 1) = 5·84 tons per square inch.

There is some doubt in this particular case about the value of the permissible buckling length, as this will be dependent upon the construction of the supports. Fig. 1 shows the general procedure. In this case a buckling length of 16″ has been assumed. On this basis

Maximum permissible reaction for unstiffened web
$= Fa \times t \times l = 5{\cdot}84 \times 0{\cdot}57 \times 16 = 53{\cdot}5$ tons.

The section is therefore amply strong for web buckling.

Web Shear.

$$\text{Ratio of depth to thickness} = \frac{21{\cdot}41}{0{\cdot}57} = 37{\cdot}6.$$

As this is less than 75, the maximum permissible shear stress of 6·5 tons per square inch may be adopted.

Allowable shear $= 6{\cdot}5 \times 24 \times 0{\cdot}57 = 89$ tons.

The section is therefore satisfactory in all respects.

Note regarding $\frac{\text{depth}}{\text{span}}$ Ratio for Crane Girders.

Although in this case, with 10 ton cranes, the ratio of 15 is satisfactory, it will not be equally satisfactory for heavier cranes. As the crane capacity increases, it will be found that the ratio of depth to span needs to be reduced. In a recent design for a gantry supporting a 175-ton crane, the writer found it necessary to use a $\frac{\text{depth}}{\text{span}}$ ratio of 7.

Floor Steelwork.

Fig. 10 gives a layout of a typical bay of floor steelwork supporting a reinforced concrete floor. An alternative design is given later for steelwork supporting an open steel floor, in order to illustrate the different procedure required.

The floor is assumed to support a super load of 2 cwts. per square foot. The R.C. floor is assumed to be 6″ thick, and haunched to the bottom flange of the floor beams. A weight of 72 lbs. per square foot, exclusive of haunching, is included for the R.C. floor, that is, 12 lbs. per square foot per inch of thickness, and an allowance of 28 lbs. per square foot is added to cover for floor finishes.

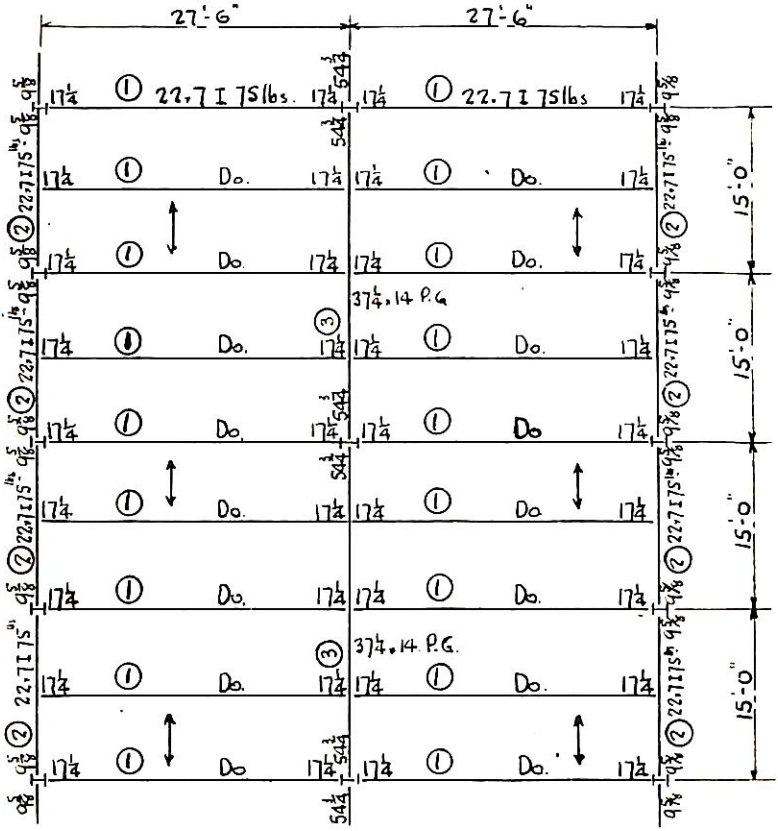
27'-6"
27'-6"
22.7 I 75lbs.
Do.
37¼, 14 P.G.
15'-0"
Loading:- Super Load 224 lbs per Sq ft
6" R.C. Floor 72 " " "
Floor Finish 28 " " "
Total 324 " " "
(Say 3 Cwts. per Sq ft = 0·15 ton)

Fig. 10—Layout of Floor Steelwork with R.C. Floor.

Load per square foot—Super load = 224 lbs.
6″ R.C. Floor = 72 lbs.
Finish = 28 lbs.
324 lbs.

Say 3 cwts. per square foot = 0·15 ton.

Floor Beams 27′-6″ Span (1).

Load = $27{\cdot}5 \times 7{\cdot}5 \times 0{\cdot}15$ = 31 tons
Own weight + haunching = Say 3·5 tons
34·5 tons

$$M \text{ at centre} = \frac{34{\cdot}5 \times 27{\cdot}5 \times 12}{8} = 1423 \text{ tons ins.}$$

It is assumed that the R.C. floor and haunching gives adequate lateral restraint for the beams, which may therefore be stressed to the maximum permissible, that is 10 tons per square inch.

F_b = 10 tons per square inch. Z required $= \frac{1423}{10} = 142{\cdot}3$ ins.^{3}

Use 22″ × 7″ × 75 lbs. R.S.J. Z given $= 152{\cdot}4$ ins.^{3}

$f_b = \frac{1423}{152{\cdot}4} = 9{\cdot}27$ tons per square inch. $R_L = R_R = 17{\cdot}25$ tons.

Side Longitudinal Beams 15′-0″ Span (2).

Central load = 17·25 tons.
Own weight + haunching = Say 2 tons.

$$M \text{ at centre} = \left\{ \left(\frac{17{\cdot}25 \times 15}{4} \right) + \left(\frac{2 \times 15}{8} \right) \right\} 12 = 821 \text{ tons ins.}$$

F_b = 10 tons per square inch. Z required $= \frac{821}{10} = 82{\cdot}1$ ins.^{3}.

For construction **use 22″ × 7″ × 75 lbs. R.S.J.** $R_L = R_R = 9{\cdot}63$ tons

If it should be necessary to keep down the weight as low as possible, a 20″ × 6½″ R.S.J. could be used for these beams without detriment to stability, but this involves a deep notch to the bottom flanges of the incoming 22 × 7 joists. Alternatively the top flange of the 20″ × 6½″ joist could be located 1″ below the level of the top flange of the 22″ × 7″ joists, giving notches of 2¾″ to both flanges of the 22″ × 7″, which is not particularly objectionable.

Centre Longitudinal Girders 30′-0″ Span (3).

Point load at centre and at quarter points = 34·5 tons.
Own weight + haunching Say = 6 tons.

$$\text{M at centre} = \left\{ \left(\frac{34{\cdot}5 \times 30}{4} \right) + (34{\cdot}5 \times 7{\cdot}5) + \left(\frac{6 \times 30}{8} \right) \right\} 12$$

$$= 6480 \text{ tons ins.}$$

For a compound beam section, $F_b = 10$ tons per square inch.

$$Z \text{ required} = \frac{6480}{10} = 648 \text{ ins.}^3.$$

Use 24″ × 7½″ × 95 lbs. R.S.J. with two 12″ × 1⅞″ flats.

$$Z \text{ given} = 666{\cdot}7 \text{ ins}^3. \quad f_b = \frac{6480}{666{\cdot}7} = 9{\cdot}72 \text{ tons per sq. in.}$$

$R_L = R_R = 34{\cdot}5 + 17{\cdot}25 + 3 = 54{\cdot}75$ tons.

An objection to this section is that the reaction is excessive and the web will require stiffening at the ends against buckling. Also deep notches nearly 4″ deep will be required to the incoming 22″ × 7″ joists. In view of these considerations it will probably be better to use a plate girder section, which will also weigh less than the compound joist section, but will of course be more expensive in fabrication. In all such problems of this kind it is necessary to balance advantages of each construction against disadvantages, and arrive at a decision in each case on its merits.

Alternative Section for Plate Girder.

$$F_b = 9{\cdot}5 \text{ tons per sq. in.} \quad \text{Nett } Z \text{ required} = \frac{6480}{9{\cdot}5} = 682 \text{ ins.}^3.$$

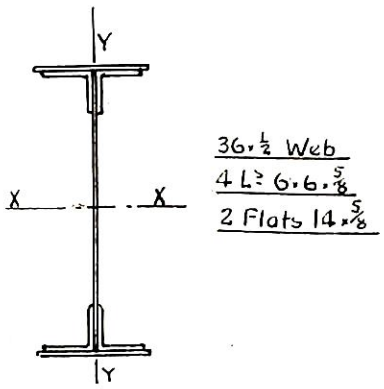

Fig. 11—Trial Section.

The trial section is given in Fig. 11, consisting of a 36″ × ½″ web plate, 6″ × 6″ × ⅝″ flange angles, and 14″ × ⅝″ flange plates.

Gross inertia of $36'' \times \frac{1}{2}''$ web $= \dfrac{\frac{1}{2} \times 36^3}{12} = 1944$ ins.^{4}.

,, ,, ,, $6'' \times 6'' \times \frac{5}{8}''$ angles $= 4\{23{\cdot}7 + (7{\cdot}11 \times 16{\cdot}3^2)\} = 7652$ ins.^{4}.

,, ,, ,, $14'' \times \frac{5}{8}''$ flange plates $= 419{\cdot}23 \times 14 = 5869$ ins.^{4}.

Total gross inertia $= 15465$ ins.^{4}.

Deduct for holes in flanges ($\frac{15}{16}''$ diameter).

$(4 \times 1\frac{1}{4} \times \frac{15}{16}'' \times 18^2) + (2 \times 1\frac{3}{4} \times \frac{15}{16} \times 13\frac{1}{2}^2) = 1519 + 598 = 2117$ ins.^{4}.

Nett inertia $= 13348$ ins.^{4}.

$$\text{Nett } Z = \frac{13348}{18{\cdot}75} = 712 \text{ ins.}^3.$$

$$f_b = \frac{6480}{712} = 9{\cdot}1 \text{ tons per sq. in.}$$

The slight reduction in section modulus due to holes in web at stiffeners or connections may be ignored.

It is necessary to check the permissible stress for the compression flange.

Gross inertia—Axis YY

$= (2 \times 114{\cdot}3) + 4\{23{\cdot}7 + (7{\cdot}11 \times 1{\cdot}96^2)\} + 0{\cdot}375$

$= 431$ ins.^{4}.

Gross area of section $= (2 \times 7) + (4 \times 7{\cdot}11) + (36 \times \frac{1}{2}) = 60{\cdot}44$ sq. ins.

$$r_{yy} = \sqrt{\frac{431}{60{\cdot}44}} = 2{\cdot}67'' \qquad r_{xx} = \sqrt{\frac{15465}{60{\cdot}44}} = 16''.$$

$$\frac{r_{xx}}{r_{yy}} = \frac{16}{2{\cdot}67} = 6 \quad \therefore \quad K_1 = 1.$$

$$F_{bc} = \frac{1000}{l/r} \times K_1 = \frac{1000 \times 2{\cdot}67}{90} \times 1$$

$= > 9{\cdot}5$ tons (maximum) per sq. in.

The section is therefore satisfactory for bending.

From an examination of permissible shear stress in Clause 20 it will be seen that the web is amply strong for shear, and this need not be checked in detail.

Rivet Pitches in Flanges.

Checking the rivets through the vertical legs of the flange angles, which is usually the criterion of design for a girder of this make up, these rivets must take the horizontal shear.

Horizontal shear per foot at ends = vertical shear per foot

$$= \frac{54{\cdot}75}{2{\cdot}5} = 21{\cdot}9 \text{ tons.}$$

Bearing value of $\frac{15}{16}''$ rivets on $\frac{1}{2}''$ web at 15 tons per sq. in. $= 7$ tons.

No. of rivets required per foot at ends $= \dfrac{21{\cdot}9}{7} =$ Say 3.

Rivets will be spaced 8″ pitch on line and staggered.

Rivets through flange plates will also be made the same pitch to line up with the web rivets, and as there will be twice as many as in the web it is not necessary to check them for strength.

Stiffeners.

Clause 47a of the original issue of the specification was not very clear, but uncertainty as to meaning has now been cleared up by the issue of Amendment No. 1. It states that bearing stiffeners are to be provided at all points of concentrated load and at reactions where the web would otherwise be overstressed, that is, overstressed in shear. The term "bearing stiffener" means one with a tight and uniform bearing on the flanges, and is thus not the same thing as a stiffener included merely to stiffen the web plate. In this example the web is amply strong enough to take the shear, and bearing stiffeners are not therefore needed. However, to explain the procedure they have been included in this case. Clause 47b states that they are to be designed as struts having an assumed design length of three-quarters of their actual length (for convenience usually taken as three-quarters of the depth over flange angles) and a portion of the web may be assumed to act with the stiffener. Further, the outstanding legs of the stiffener must have a size of at least 2″ plus one-thirtieth of the girder depth, and must extend as nearly as practicable to the edge of the flange angles. Finally they are to have a sufficient area of section outside the roots of the flange angles to transmit the whole of the reaction in bearing at a stress of 12 tons per square inch, while the rivets connecting the stiffeners to the girder web must resist the full reaction with a pitch not exceeding 6″.

Minimum length of outstanding legs $= 2 + \dfrac{36}{30} = 3{\cdot}2$ ins.

Area required in outstanding legs (outside flange angle roots)

$$= \frac{54{\cdot}75}{12} = 4{\cdot}56 \text{ sq. ins.}$$

2 angles $5'' \times 4'' \times \frac{1}{2}''$, with 5″ legs outstanding, will be used.

Area given in outstanding legs (outside flange angle roots)

$$= 2 \times 4{\cdot}52 \times \tfrac{1}{2} = 4{\cdot}52 \text{ sq. ins.}$$

This will be assumed satisfactory.

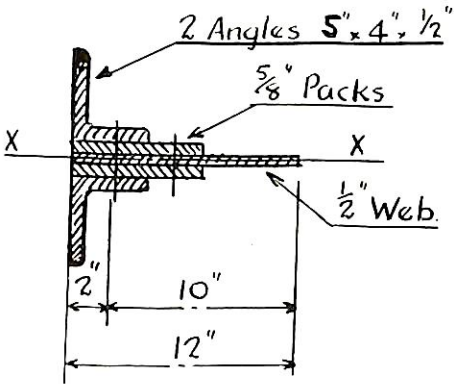

Fig. 12.

End Stiffeners as Struts (Fig. 12).

A portion of the web is assumed to act with the stiffeners in accordance with Clause 47b of the specification. The packings are assumed to be ineffective, although as they extend outside the stiffeners they will certainly give some slight assistance.

$I_{xx} = 2\{10{\cdot}29 + (4{\cdot}25 \times 2{\cdot}44^2)\} + 0{\cdot}125 = 71$ ins.^{4}.

Area of section $= (2 \times 4{\cdot}25) + 6 = 14{\cdot}5$ sq. ins.

$$r_{xx} = \sqrt{\frac{71}{14{\cdot}5}} = 2{\cdot}21 \text{ ins.}$$

$$l/r = \frac{36 \times \frac{3}{4}}{2{\cdot}21} = 12{\cdot}2. \quad \therefore F_c \text{ (Table 1)} = 8{\cdot}4 \text{ tons per sq. in.}$$

Permissible load $= 8{\cdot}4 \times 14{\cdot}5 = 122$ tons.

Section is therefore satisfactory as a strut.

Rivets Through End Stiffeners.

Bearing value of $\frac{15}{16}$" rivets on $\frac{1}{2}$" web at 15 tons per sq. in. $= 7$ tons.

$$\text{No. of rivets required} = \frac{54{\cdot}75}{7} = 8.$$

Rivets Through Seating Bracket.

Single shear value of $\frac{15}{16}$" rivets at 6 tons per sq. in. $= 4{\cdot}14$ tons.

$$\text{No. of rivets required} = \frac{54{\cdot}75}{4{\cdot}14} = 14.$$

Fig. 13 shows detail of connection.

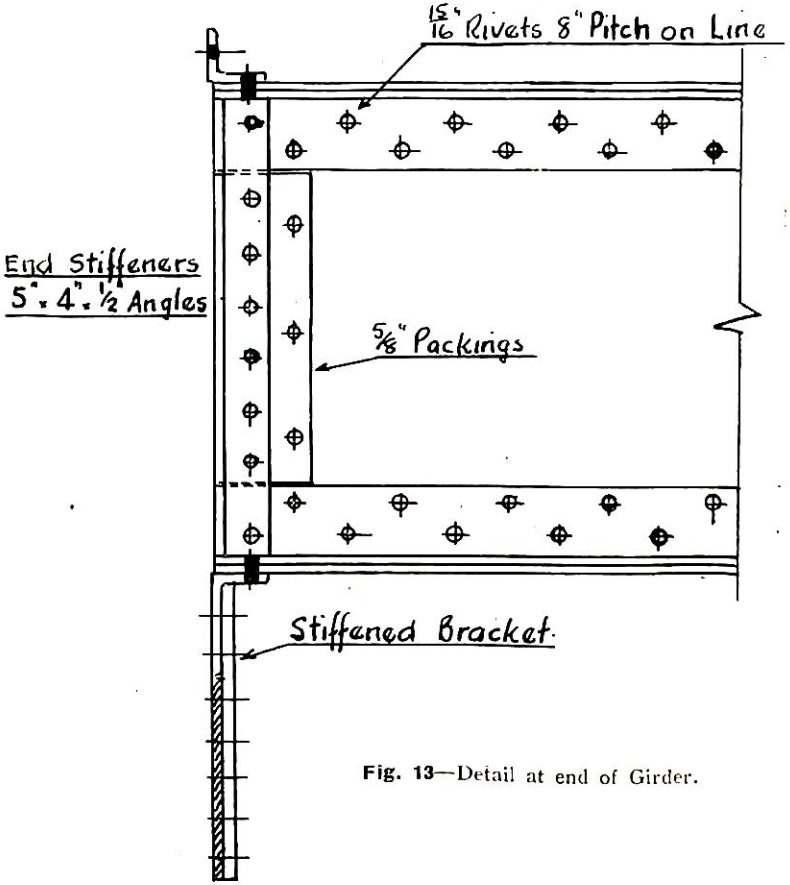

Fig. 13—Detail at end of Girder.

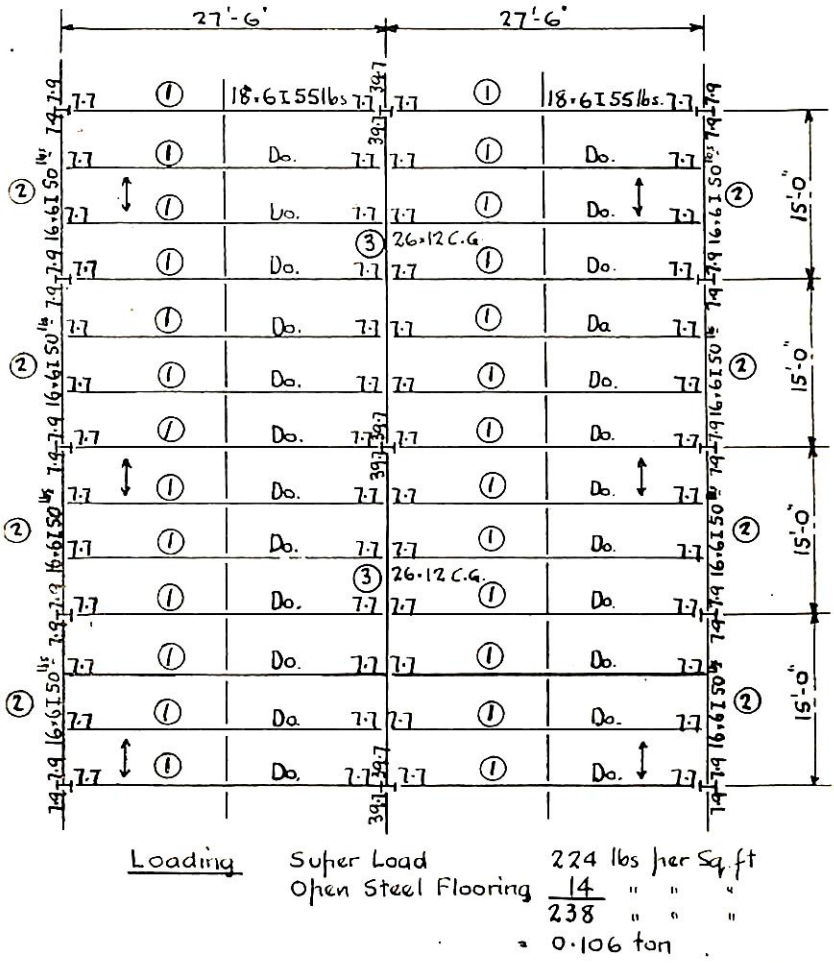

Fig. 14—Layout of Floor Steelwork with Open Steel Flooring.

Intermediate Stiffeners.

Clause 20b of the specification, as re-written in Amendment No. 1, states that intermediate stiffeners designed in accordance with Clause 47, are to be provided throughout the length of the girder at distances apart not exceeding one-and-a-half times the depth of the girder in cases where the thickness of the web is less than one seventy-fifth of the clear distance between the flange angles. This condition does not apply in this example. Nevertheless it seems desirable for good construction to introduce stiffeners at the connection of incoming beams, even though the web is not over-stressed at these points.

Fig. 14 gives an alternative layout for the same floor with a covering of open steel flooring in place of the reinforced concrete floor. Floor beam centres have been reduced to 5′-0″ to suit permissible loads on flooring, and as the open steel floor does not give adequate lateral restraint for the floor beams additional light tie beams are introduced at mid span to reduce the unsupported beam length. These tie beams are connected to the webs of the floor beams, and as high up as possible to restrain the top flanges.

Load per Square Foot.

Super load	=	224 lbs.
Open steel flooring	=	14 lbs.
		238 lbs. = 0·106 tons.

Floor Beams 27′-6″ Span (1).

Load = 27·5 × 5 × 0·106	=	14·6 tons
Own weight	=	0·8 tons
		15·4 tons

$$M \text{ at centre} = \frac{15{\cdot}4 \times 27{\cdot}5 \times 12}{8} = 635 \text{ tons ins.}$$

Try 18″ × 6″ × 55 lbs. R.S.J. with centre tie.

$$\frac{r_{xx}}{r_{yy}} = \frac{7{\cdot}21}{1{\cdot}21} = 6. \quad \therefore \ K_1 = 1.$$

$$F_{bc} = \frac{1000}{l/r} \times K_1 = \frac{1000 \times 1{\cdot}21}{165} \times 1 = 7{\cdot}33 \text{ tons per sq. in.}$$

$$Z \text{ required} = \frac{635}{7{\cdot}33} = 86{\cdot}6 \text{ ins.}^3 \quad Z \text{ given} = 93{\cdot}5 \text{ ins.}^3.$$

Section is satisfactory. $f_b = \frac{635}{93{\cdot}5} = 6{\cdot}8$ tons per sq. in.

$R_L = R_R = 7{\cdot}7$ tons.

Side Longitudinal Beams 15′-0″ Span (2).

Point load 5′-0″ from each end = 7.7 tons.
Own weight = 0·4 tons.

$$M \text{ at centre} = \left\{(7{\cdot}7 \times 5) + \left(\frac{0{\cdot}4 \times 15}{8}\right)\right\} 12 = 471 \text{ tons ins.}$$

For construction **try 18″ × 6″ × 55 lbs R.S.J.** as for beams (1).

$$F_{bc} = \frac{1000}{60} \times 1{\cdot}21 = 10 \text{ tons per sq. in. maximum.}$$

$$Z \text{ required} = \frac{471}{10} = 47{\cdot}1 \text{ ins.}^3 \quad Z \text{ given} = 93{\cdot}5 \text{ ins.}^3.$$

Section is too strong and can hardly be justified on grounds of uniformity.

Try 16″ × 6″ × 50 lbs. R.S.J.

$$\frac{r_{xx}}{r_{yy}} = \frac{6{\cdot}48}{1{\cdot}24} = 5{\cdot}2 \quad \therefore \ K_1 = 1.$$

$$F_{bc} = \frac{1000 \times 1{\cdot}24}{60} \times 1 = 10 \text{ tons per sq. in. maximum.}$$

Z required = 47·1 ins.^{3}. Z given = 77·3 ins.^{3}.

Section is still too strong for loads but it is undesirable to reduce it further owing to the depth of notches required in incoming 18″ × 6″ joists.

Use 16″ × 6″ × 50 lbs. R.S.J. $f_b = \frac{471}{77{\cdot}3} = 6{\cdot}1$ tons per sq. in.

$R_L = R_R = 7{\cdot}7 + 0{\cdot}2 = 7{\cdot}9$ tons.

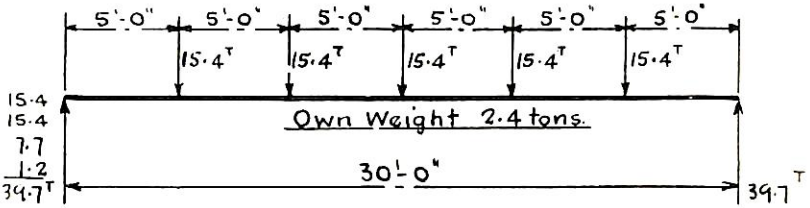

Fig. 15—Loading Diagram for Beams (3) in Fig. 14.

Centre Longitudinal Beams 30′-0″ Span (3).

Loading diagram is given in Fig. 15.

$$M \text{ at centre} = \left\{\left(\frac{15{\cdot}4 \times 30}{4}\right) + 15{\cdot}4\,(5+10) + \left(\frac{2{\cdot}4 \times 30}{8}\right)\right\} 12$$

$$= 4266 \text{ tons ins.}$$

F_{bc} = 10 tons per sq. in. Z required = $\frac{4266}{10}$ = 426·6 ins.^{3}.

For flexure **use 24″ × 7½″ × 95 lbs. R.S.J. + 2 flats 12″ × 1″.**

Z given = 442·2 ins.^{3}. $f_b = \frac{4266}{442·2}$ = 9·65 tons per sq. in.

$R_L = R_R$ = 39·7 tons.

Check Section for Buckling.

It is assumed that the supporting bracket gives a stiff bearing 1½″ long, so that the effective buckling length of the web in accordance with the procedure shown in Fig 1 is 1·5 + 12 = 13½″.

Slenderness ratio of web = $\frac{d\sqrt{3}}{t} = \frac{21·41 \times 1·732}{0·57}$ = 65

∴ F_a (Table 1) = 5·84 tons per sq. in.

Permissible buckling value for unstiffened web

= 5·84 × 13·5 × 0·57 = 44·9 tons.

Actual reaction is 39·7 tons, and the web does not therefore require stiffening against buckling.

It is also necessary to check the web for bearing value in accordance with the last paragraph of Clause 47a, which states that the direct bearing stress delivered to that part of the web immediately adjoining the roof fillets must not exceed 12 tons per square inch, assuming a dispersion of load through the parts at 30 degrees to the horizontal. This condition is set down in Fig. 16.

Clear depth of beam between fillets = 20·22″.

Distance from underside of beam to top of fillet

= $\frac{24 - 20·22}{2}$ = 1·89″.

Bearing length of joist = $\frac{1·89}{\tan 30°}$ = 3·27″.

Total bearing length = 3·27 + 1·5 = 4·77″.

Maximum permissible bearing value of web

= 4·77 × 0·57 × 12 = 32·63 tons.

This is less than the actual reaction of 39·7 tons and the web will have to be stiffened over the end bracket. Two angles 3½″ × 3″ × ½″ will be included with 3″ outstanding legs, the outstanding legs being located immediately above the stiffened bearing. Alternatively the bearing length of the web can be increased by introducing a ⅝″ thick packing between the seating and the beam.

Increase in bearing length = $\frac{0·625}{\tan 30°}$ = 1·08″.

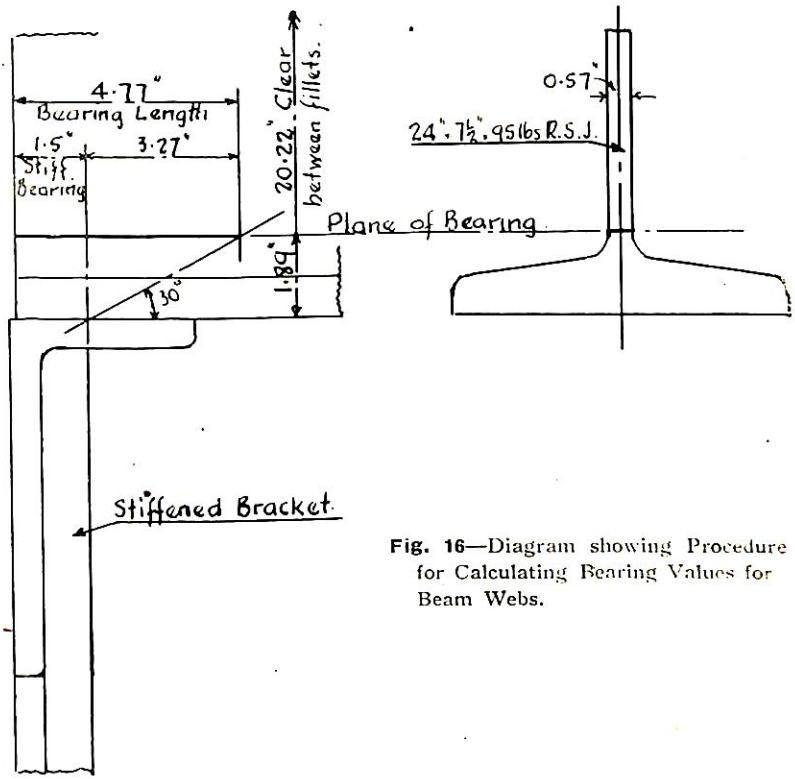

Fig. 16—Diagram showing Procedure for Calculating Bearing Values for Beam Webs.

Total bearing length = 4·77 + 1·08 = 5·85".
Maximum permissible bearing value of web = 5·85 × 0·57 × 12
= 40 tons.
On this basis the web has sufficient strength without stiffeners.

Curtailment of Flange Plates.

In cases of this kind, particularly as Clause 48c allows some latitude in fixing exact lengths of plates, it is not necessary to draw a bending moment diagram in order to find the position of plate curtailment. A simple procedure is as follows :—

Z of beam alone	= 211 ins.^{3}
Actual f_c	= 9·65 tons per sq. in.
B.M. taken by beam = 211 × 9·65	= 2036 tons ins.
	= 169·7 tons ft.
Reaction	= 39·7 tons.

$$\therefore \text{ Distance from reaction to flange cut off } = \frac{169{\cdot}7}{39{\cdot}7}$$

$$= 4{\cdot}27 \text{ feet.}$$

Nett length of plate $= 30 - (2 \times 4{\cdot}27) = 21{\cdot}46$ feet (say 21′-6″).

Specification states that plates shall be extended beyond this theoretical point of cut-off a sufficient distance to accommodate three rivets in line at each end.

Rivet Pitch in Flanges.

Rivets must resist the horizontal shear in the plane of the joist flanges, in accordance with the following formula :—

$$\text{Horizontal shear per foot } = \frac{12\ VG}{I}$$

where V = vertical shear at the point in question.
G = first moment of the flange plate about the neutral axis.
I = moment of inertia of the complete section.

In this case

$$\text{horizontal shear per foot } = \frac{12 \times 39{\cdot}7 \times 12 \times 1 \times 12{\cdot}5}{5749}$$

$$= 12{\cdot}4 \text{ tons.}$$

Single shear value of $\frac{15}{16}$″ rivets at 6 tons per sq. in. $= 4{\cdot}14$ tons.

$$\text{No. of rivets required per foot in each flange } = \frac{12{\cdot}4}{4{\cdot}14} = 3.$$

On this basis rivets could be 8″ pitch on line and staggered.

Specification states, however, that for the compression flange the straight line pitch is not to exceed 6″, and for uniformity a 6″ pitch will be used for both flanges.

In connection with the alternative layout shown in Fig. 14, it might be noted that if two lines of tie beams are included between the floor beams, located at the third points instead of one line at the centre, the floor beam section could be reduced from $18 \times 6 \times 55$ lbs. to $16 \times 6 \times 50$ lbs. owing to the reduction in centres of lateral restraint. As against this saving in weight, there would be an increase in weight due to the additional tie beams and end cleats, and also an increase in costs of workmanship. These are typical points which the designer must endeavour to balance against each other, in order to arrive at an economical construction.

Stanchion for Office Building.

Fig. 17 gives a line diagram for a side wall stanchion to a three-storey building. The heights given from floor to floor are centres of incoming beam connections, in accordance with the specification. Beams connecting to the stanchion flanges are wall beams and are

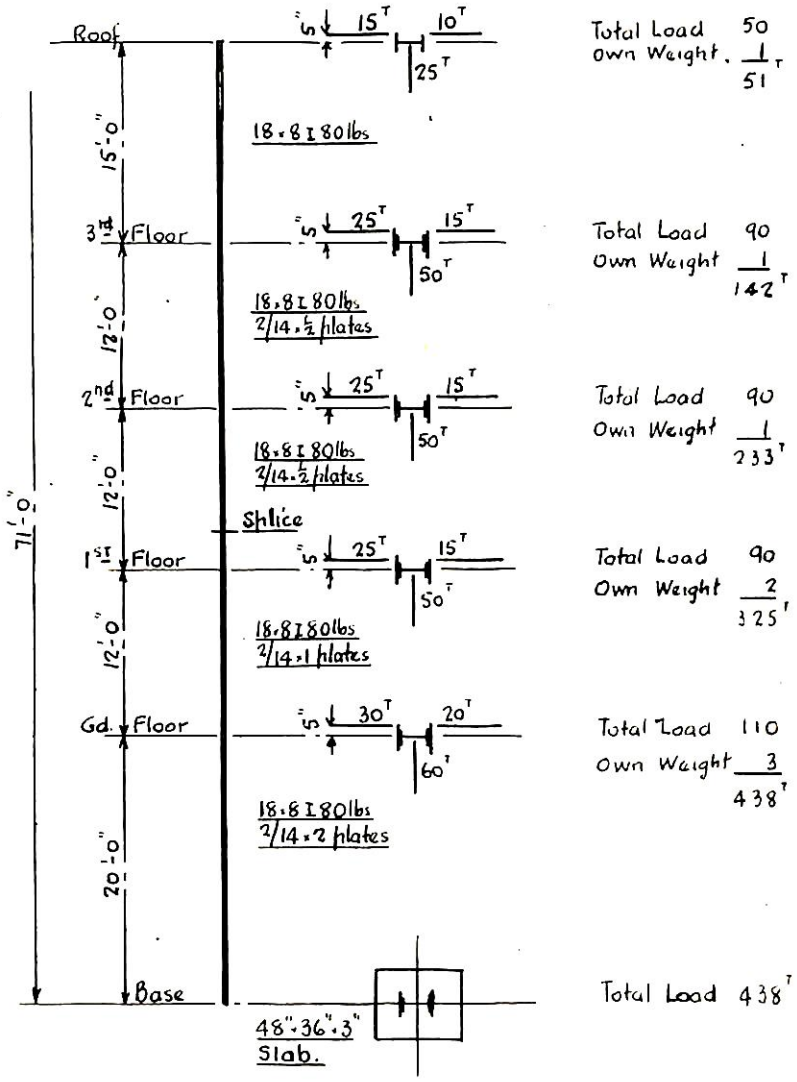

Fig. 17—Diagram of Stanchion Loads for Office Building.

all 5″ eccentric. The girder connecting to the web at ground floor level is a plate girder. Beams connecting to the web at first, second, and third floors are 24″ × 7½″ joist compounds, and at roof level 22″ × 7″ joist. The incoming girder therefore requires a clear width in the web of the stanchion of at least 14″. The rivet heads inside the stanchion flanges require a further 1″, so that the minimum section suitable as a basis joist is 18″ × 7″ or 18″ × 8″. To assist riveting and to keep the radius of gyration about the weaker axis as high as possible an 18″ × 8″ joist will be used as basis, with 14″ flange plates to give four lines of rivets for flange connections.

The effective lengths of the various sections of the stanchion will be fixed up in accordance with the sketches shown in Appendix E of the specification. On this basis the length multiplier for the lower length and for the top length will be taken as 0·85, bearing in mind that the base is a slab. For the other length the multiplier will be taken as 0·7 Effective lengths will therefore be as follows :

Lower length	20 × 0·85	=	17′-0″	= 204″
Top length	15 × 0·85	=	12·75′	= 153″
All other lengths	12 × 0·7	=	8·4′	= 101″

As the beams connecting to the flanges are eccentric, the bending moments about the weaker axis produced by these reactions will be in opposition to the bending moment from the web connection, and the nett moment about the weaker axis will be the difference between these two moments. The specification also provides for the bending moments at each floor to be divided equally between the lengths above and below the floor, provided that the moment of inertia of either length divided by its actual length does not exceed one-and-a-half-times the corresponding value for the other length.

This proviso does not affect this design, and in the following calculations the moments at the floors have been divided equally. In any case, where moments are small, the manner of sharing the moments is not usually of very great importance. For the roof length of course the total moment at the roof must be taken into consideration.

The general design procedure is to be in accordance with Clause 22 of the specification for combined stresses.

Lower Length—Basement to Ground Floor.

Effective length = 20 × 0·85 = 17′-0″ = 204″.
Load = 438 tons.

Try 18 × 8 × 80 lbs. R.S.J. with two 14″ × 2″ plates.

Area of section = 79·53 sq. ins.

Z_{xx} =	628·3 ins.³.	Z_{yy} =	140·6 ins.³.
r_{xx} =	9·32 ins.	r_{yy} =	3·52 ins.

$\frac{r_{xx}}{r_{yy}} = \frac{9{\cdot}32}{3{\cdot}52} = 2{\cdot}6.$ $\therefore$ $K_1 = 1{\cdot}5$

$l/r = \frac{204}{3{\cdot}52} = 58.$ $\therefore$ F_a (Table 1) $= 6{\cdot}19$ tons p. sq. in.

f_a (actual compressive stress) $= \frac{438}{79{\cdot}53} = 5{\cdot}51$ tons per sq. in.

Assumed arm of eccentricity—Axis XX $= 9'' + 2'' + 1'' = 12''$.
Assumed arm of eccentricity—Axis YY $= 2''$ (see Clause 37).

$M_{xx} = \frac{10 \times 12}{2} = 60$ tons ins. $M_{yy} = \frac{(50 \times 5) - (60 \times 2)}{2}$

$= 65$ tons ins.

$f_{bcxx} = \frac{60}{628{\cdot}3} = 0{\cdot}1$ ton/sq. in. $f_{bcyy} = \frac{65}{140{\cdot}6} = 0{\cdot}46$ ton/sq.in.

Total $f_{bc} = 0{\cdot}1 + 0{\cdot}46 = 0{\cdot}56$ ton per sq. in.

$F_{bc} = \frac{1000}{l/r} \times K_1 = \frac{1000}{58} \times 1{\cdot}5 = > 10$ tons (maximum) per sq. in.

$\frac{f_a}{F_a} + \frac{f_{bc}}{F_{bc}} = \frac{5{\cdot}51}{6{\cdot}19} + \frac{0{\cdot}56}{10} = 0{\cdot}96$

Factor 0·96 is less than unity and section is therefore satisfactory.

Use 18 × 8 × 80 lbs. R.S.J. with two 14″ × 2″ plates.

Second Length. Ground Floor to First Floor.

Effective length $= 12 \times 0{\cdot}7 = 8{\cdot}4$ feet $= 101''$.
Load $= 325$ tons.

Try 18 × 8 × 80 lbs. R.S.J. with two 14″ × 1″ plates.

Area of section $= 51{\cdot}53$ sq. ins.
$Z_{xx} = 382{\cdot}1$ ins.^{3} $Z_{yy} = 75{\cdot}3$ ins.^{3}.
$r_{xx} = 8{\cdot}61$ ins. $r_{yy} = 3{\cdot}2$ ins.

$l/r = \frac{101}{3{\cdot}2} = 31{\cdot}5.$ $\therefore$ $F_a = 7{\cdot}46$ tons per sq. in.

$f_a = \frac{325}{51{\cdot}53} = 6{\cdot}31$ tons per sq. in.

$M_{xx} = \frac{10 \times 11}{2} = 55$ tons ins. $M_{yy} = \frac{(40 \times 5) - (50 \times 2)}{2}$

$= 50$ tons. ins.

$f_{bcxx} = \frac{55}{382{\cdot}1} = 0.14$ ton/sq. in. $f_{bcyy} = \frac{50}{75{\cdot}3}$

$= 0{\cdot}66$ ton/sq. in.

Total $f_{bc} = 0{\cdot}14 + 0{\cdot}66 = 0{\cdot}80$ ton per sq. in.

$$\frac{f_a}{F_a} + \frac{f_{bc}}{F_{bc}} = \frac{6{\cdot}31}{7{\cdot}46} + \frac{0{\cdot}80}{10} = 0{\cdot}93.$$

Factor 0·93 is less than unity and section is satisfactory.

Use 18 × 8 × 80 lbs. R.S.J. with two 14″ × 1″ plates.

Third Length—1st Floor to 2nd Floor.

Effective length = $12 \times 0{\cdot}7 = 8{\cdot}4$ feet = 101 ins.
Load = 233 tons.

Try 18 × 8 × 80 lbs. R.S.J. with two 14″ × ½″ plates.

Area of section = 37·53 sq. ins.
$Z_{xx} = 262{\cdot}1$ ins.^{3}. $Z_{yy} = 42{\cdot}6$ ins.^{3}.
$r_{xx} = 8{\cdot}15$ ins. $r_{yy} = 2{\cdot}82$ ins.

$l/r = \frac{101}{2{\cdot}82} = 36.$ $\therefore$ $F_a = 7{\cdot}25$ tons per sq. in.

$f_a = \frac{233}{37{\cdot}53} = 6{\cdot}21$ tons per sq. in.

$M_{xx} = \frac{10 \times 10\frac{1}{2}}{2} = 52{\cdot}5$ tons ins. $M_{yy} = \frac{(40 \times 5) - (50 \times 2)}{2}$

$= 50$ tons. ins.

$f_{bcxx} = \frac{52{\cdot}5}{262{\cdot}1} = 0{\cdot}2$ ton/sq. in. $f_{bcyy} = \frac{50}{42{\cdot}6}$

$= 1{\cdot}17$ tons/sq. in.

Total $f_{bc} = 0{\cdot}2 + 1{\cdot}17 = 1{\cdot}37$ tons per sq. in.

$$\frac{f_a}{F_a} + \frac{f_{bc}}{F_{bc}} = \frac{6{\cdot}21}{7{\cdot}25} + \frac{1{\cdot}37}{10} = 0{\cdot}99.$$

Factor 0·99 is less than unity, and section is just strong enough.

Use 18 × 8 × 80 lbs. R.S.J. with two 14″ × ½″ plates.

Fourth Length—2nd Floor to 3rd Floor.

Effective length = $12 \times 0{\cdot}7 = 8{\cdot}4$ feet = 101 ins.
Load = 142 tons.

Try 18 × 8 × 80 lbs. R.S.J. without flange plates.

Area of section = 23·53 sq. ins.
Z_{xx} = 143·56 ins.^{3}. Z_{yy} = 17·36 ins.^{3}.
r_{xx} = 7·41 ins. r_{yy} = 1·72 ins.

$$l/r = \frac{101}{1{\cdot}72} = 59. \quad \therefore \quad F_a = 6{\cdot}14 \text{ tons per sq. in.}$$

$$f_a = \frac{142}{23{\cdot}53} = 6{\cdot}03 \text{ tons per sq. in.}$$

$$M_{xx} = \frac{10 \times 10}{2} = 50 \text{ tons ins.} \quad M_{yy} = \frac{(40 \times 5) - (50 \times 2)}{2} = 50 \text{ tons ins.}$$

$$f_{bcxx} = \frac{50}{143{\cdot}56} = 0{\cdot}35 \text{ ton/sq. in.} \quad f_{bcyy} = \frac{50}{17{\cdot}36} = 2{\cdot}88 \text{ tons/sq. in.}$$

Total f_{bc} = 0·35 + 2·88 = 3·23 tons per sq. in.

$$\frac{f_a}{F_a} + \frac{f_{bc}}{F_{bc}} = \frac{6{\cdot}03}{6{\cdot}14} + \frac{3{\cdot}23}{10} = 1{\cdot}3$$

Factor 1·3 is more than unity, and section is therefore too weak. The $\frac{1}{2}$" plates from the length below will be extended up through this length, and will obviously give sufficient strength without checking in detail.

Use 18 × 18 × 80 lbs. R.S.J. with two 14" × $\frac{1}{2}$" plates.

Top Length—3rd Floor to Roof.

Effective length = 15 × 0·85 = 12·75 feet = 153 ins.
Load = 51 tons.

Try 18 × 8 × 80 lbs. R.S.J without flange plates.

Area of section = 23·53 sq. ins.
Z_{xx} = 143·56 ins.^{3}. Z_{yy} = 17·36 ins.^{3}.
r_{xx} = 7·41 ins. r_{yy} = 1·72 ins.

$$l/r = \frac{153}{1{\cdot}72} = 89 \quad \therefore \quad F_a = 4{\cdot}67 \text{ tons per sq. in.}$$

$$f_a = \frac{51}{23{\cdot}53} = 2{\cdot}12 \text{ tons per sq. in.}$$

$$M_{xx} = 5 \times 10 = 50 \text{ tons ins.} \quad M_{yy} = (25 \times 5) - (25 \times 2) = 75 \text{ tons ins.}$$

$$f_{bcxx} = \frac{50}{143{\cdot}56} = 0{\cdot}35 \text{ ton/sq. in.} \quad f_{bcyy} = \frac{75}{17{\cdot}36} = 4{\cdot}33 \text{ tons/sq. in.}$$

Total f_{bc} = 0·35 + 4·33 = 4·68 tons per sq. in.

$$\frac{f_a}{F_a} + \frac{f_{bc}}{F_{bc}} = \frac{2{\cdot}12}{4{\cdot}67} + \frac{4{\cdot}68}{10} = 0{\cdot}92$$

Factor 0·92 is less than unity and section is satisfactory.

Use 18 × 8 × 80 lbs. R.S.J.

The stanchion is thus built up with 18″ × 8″ basis joist, 14″ × $\frac{1}{2}$″ plates on each flange up to 3rd floor, additional 14″ × $\frac{1}{2}$″ plates up to 1st floor, and further 14″ × 1″ plates up to ground floor. In all cases the flange plates will be continued up to the top of the incoming floor beam connections.

Rivet Pitches in Shaft.

Rivets will be made $\frac{7}{8}$″ nominal diameter in $\frac{15}{16}$″ holes. Clause 42a of the specification states that the straight line pitch is not to exceed 6″. Clause 42b states that at the ends (in this case at the base) the pitch is not to exceed 4″ for a length of one-and-a-half times the width or depth of the member, whichever is the greater. That means in this case for a distance of 2′-9″ from the base. The incoming floor beams have non-restrained connections, and Clause 42b also requires that a 4″ pitch is to be used for a similar distance below the beam connections.

Stanchion Base.

As the load is vertical and axial at the base, a slab base will be suitable. A sketch of the base is given in Fig. 18. It is assumed that the concrete foundation can withstand a safe bearing pressure of not more than 40 tons per square foot.

Load on slab = 438 tons

$$\text{Area required} = \frac{438}{40} = \text{say } 11 \text{ sq. ft.}$$

For convenience and to suit standard sizes the slab will be made 4′-0″ × 3′-0″. Generally in fixing up slab sizes it is desirable to keep dimensions A and B in Fig. 18 as nearly equal as possible.

The formula for finding the thickness of the slab is given in Clause 50c of the specification.

$$t = \sqrt{\frac{3w}{Fb}\left(A^2 - \frac{B^2}{4}\right)}$$

where t = slab thickness in inches.
A = greater projection of plate beyond the stanchion, in inches.
B = lesser projection of plate beyond the stanchion in inches.

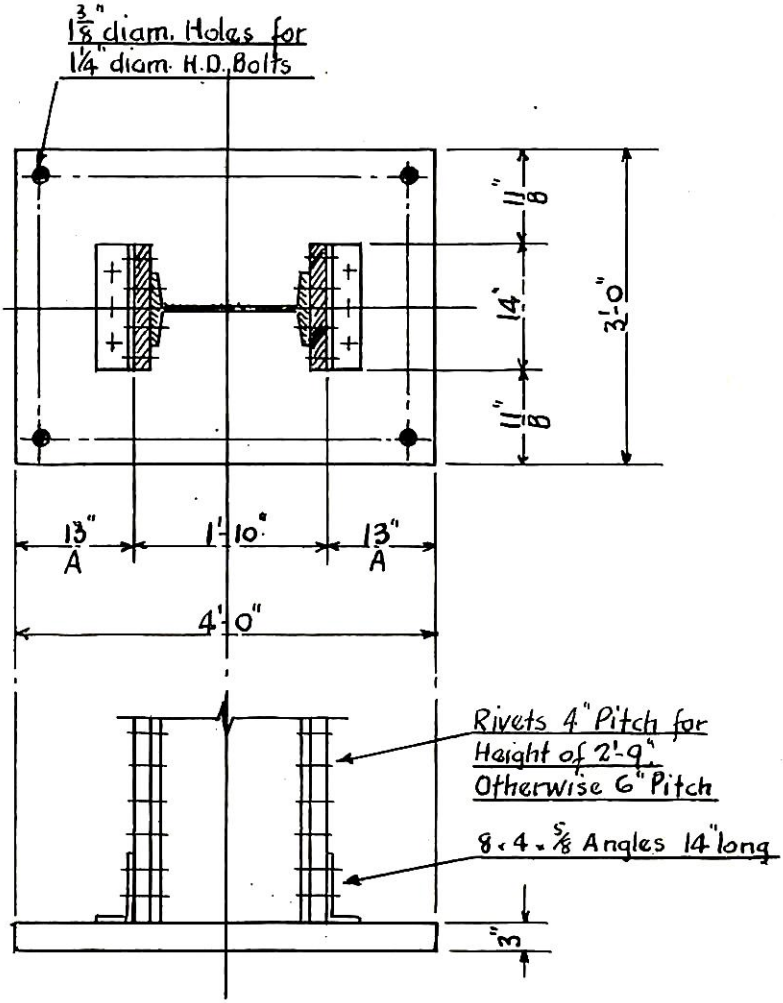

Fig. 18—Slab Base for Stanchion.

w = pressure on base in tons per sq. in.
Fb = Permissible bending stress in the steel in tons per sq. in. given in Clause 19c
= 12 tons per sq. in.

In this example $w = \frac{438}{48 \times 36} = 0{\cdot}253$ ton per sq. in

$A = 13''$
$B = 11''$

$$t = \sqrt{\frac{3 \times 0{\cdot}253}{12}\left(13^2 - \frac{11^2}{4}\right)} = \text{Say } 3 \text{ ins.}$$

Slab will be machined on top, and the stanchion shaft will be machined at the end to give a perfect bearing. It is not necessary to machine the underside of the slab, sitting on a concrete foundation. A space of about 2" will be left under the slab for grouting in, and any unevenness in the slab surface will in fact assist in binding the stanchion to the foundation.

Holding down bolts will be required, purely as positioning pins, and these will be made $1\frac{1}{4}''$ diameter and 1'-6" long with 6" square washer plates at the bottom.

Grillages for Stanchions.

In order to explain the procedure for grillage design, an alternative construction for the stanchion base is shown in Fig. 19.

Clause 27 of the specification states that all working stresses in grillage beams may be increased by one-third providing the requirements for encasing laid down in this clause are complied with. It may also be assumed that the grillage joists and concrete filling between them forms a solid homogeneous mass, so that the effective bearing area in the foundation may be derived from the overall size of the grillage, that is, 4'-6" × 2'-6". The width of 2'-6" has been fixed to suit the size of the stanchion shaft and base cleats and the 1'-10" dimension to equalise dimensions A and B.

Design of Slab.

$$\text{Area} = 30 \times 22 = 660 \text{ sq. ins.}$$

$$w = \frac{438}{660} = 0{\cdot}664 \text{ tons.}$$

$$t = \sqrt{\frac{3w}{12}\left(A^2 - \frac{B^2}{4}\right)}$$

$$= \sqrt{\frac{3 \times 0{\cdot}664}{12}\left(4^2 - \frac{4^2}{4}\right)} = 1{\cdot}4 \text{ ins.}$$

Say $1\frac{1}{2}''$ thick.

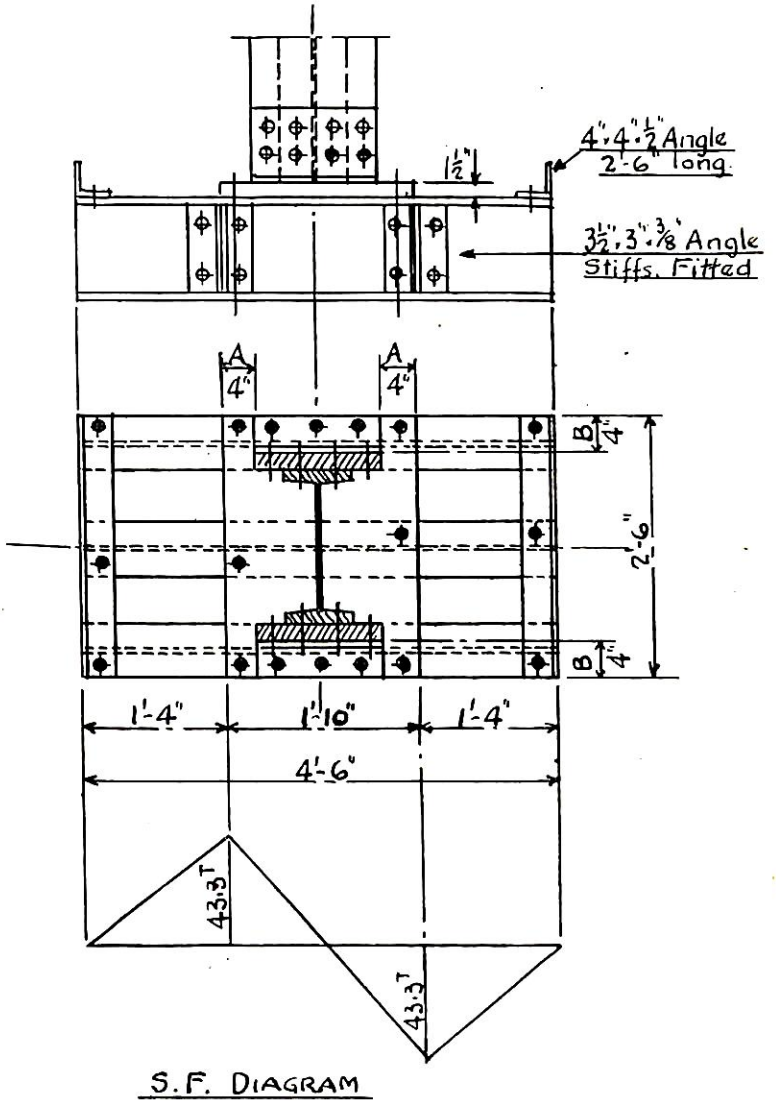

Fig. 19—Detail of Grillage Base.

Slab will be machined both top and bottom to give perfect bearing on shaft and on grillage.

Design of Grillage.

Load = 438 tons.

Permissible pressure on foundation = 40 tons per sq. ft.

Area required in grillage = $\frac{438}{40}$ = Say 11 sq. ft.

Area given = $4{\cdot}5 \times 2{\cdot}5$ = 11·25 sq. ft.

Load per beam = $\frac{438}{3}$ = 146 tons.

Load per foot of beam = $\frac{146}{4{\cdot}5}$ = 32·5 tons.

Maximum shear (at edge of slab) = $32{\cdot}5 \times 1{\cdot}33$ = 43·3 tons.

M at centre of beam = area of shear force diagram.

$$= \frac{43{\cdot}3 \times 2{\cdot}25}{2} \times 12 = 585 \text{ tons ins.}$$

F_{bc} = $10 + 33\frac{1}{3}\%$ = 13·33 tons per sq. in.

Z required = $\frac{585}{13{\cdot}33}$ = 44 ins.^{3}.

12 × 6 × 44 lbs. R.S.J. is sufficiently strong for flexure.

Check for Shear.

Actual shear stress = $\frac{43{\cdot}3}{12 \times 0{\cdot}4}$ = 9 tons per sq. in.

Allowable shear stress = $6{\cdot}5 + 33\frac{1}{3}\%$ = 8·7 tons per sq. in.

Beams are therefore slightly too weak in shear.

Check for Buckling.

Ratio of slenderness of web = $\frac{d\sqrt{3}}{t} = \frac{10{\cdot}12 \times 1{\cdot}732}{0{\cdot}4}$ = 44

F_a (Table 1) = $6{\cdot}86 + 33\frac{1}{3}\%$ = 9·15 tons per sq. in.

Allowable buckling load per inch run = $9{\cdot}15 \times 0{\cdot}4 \times 1 = 3{\cdot}66$ tons.

Assumed buckling length = width of slab + depth of beam

= 22″ + 12″ = 34″.

Maximum allowable load per beam for buckling

= $3{\cdot}66 \times 34$ = 124·44 tons.

Actual load per beam is 146 tons, so that web is also too weak in buckling.

Check for Bearing Value of Web.

It is also necessary to check whether the web area immediately adjoining the flange fillets is capable of transmitting the load of 146 tons per beam. The procedure is as given in the last paragraph of Clause 47a of the specification, and as already illustrated in Fig. 16.

Depth of web between fillets = 9·12".

$$\text{Distance from outside of flange to end of fillet} = \frac{12 - 9{\cdot}12}{2} = 1{\cdot}44".$$

$$\text{Assumed bearing length} = \text{width of slab} + \frac{(2 \times 1{\cdot}44)}{\tan 30^\circ}$$

$$= 22 + \frac{2{\cdot}88}{\tan 30^\circ} = 27".$$

Permissible bearing stress = $12 + 33\frac{1}{3}\%$ = 16 tons per sq. in.

Permissible load in bearing = $16 \times 27 \times 0{\cdot}4$ = 172·8 tons.

On this basis the webs are strong enough in bearing, as the load delivered to each web is 146 tons. However, the writer is very doubtful whether it is safe to assume the permissible bearing stress may be increased by $33\frac{1}{3}\%$ even though Clause 27 of the specification states clearly that stresses in grillage beams may be increased by this amount. The reason for the increased stresses is the assistance provided by the concrete encasement, but it seems likely that the greater part of the load will be delivered to the web before commencing to disperse, and if this is so the bearing stress should be taken as 12 tons instead of 16 tons.

On this basis, permissible load in bearing = $12 \times 27 \times 0{\cdot}4$
= 129·6 tons.

This is less than the actual load per beam of 146 tons, and in view of the fact that the webs are also slightly weak in shear and buckling, it is desirable to include fitted stiffeners under the edges of the base slabs.

Stanchion Splice.

Owing to the length of the stanchion it will be necessary to include a splice, and this can conveniently be located above the first floor. As the load from the floors above will all be axial at the splice, and the bending moment at first floor level due to eccentric connections is very small, it will be in order to make the splice in accordance with Clause 50b of the specification. This states that when the ends are machined for bearing they shall be sufficiently spliced to hold the two lengths together, the effective length of the covers on each side of the joint being not less than the width of the flange. A detail of a suitable splice is shown in

Fig. 20, and needs no comment, beyond observing that as all the vertical load is assumed to be transmitted by bearing, the rivets provided for the joint will be more than ample to take care of any small bending moment resulting from eccentricity of connection on the floor below.

Cased Stanchions.

It is convenient at this stage to consider the variations in procedure for stanchions encased in concrete in accordance with Clause 18b of the specification. There will be a small additional load due to the weight of the casing, and the value of the least radius of gyration will be increased to $0{\cdot}2\ (b+4)$, where b is the width of the stanchion.

In this case, up to third floor level, the radius of gyration about the weaker axis will be $0{\cdot}2\ (14+4) = 3{\cdot}6''$. This value is very little more than for the uncased section, so that although the cased stanchion will take more load than the uncased condition, the increase will not be sufficient to enable the section to be reduced.

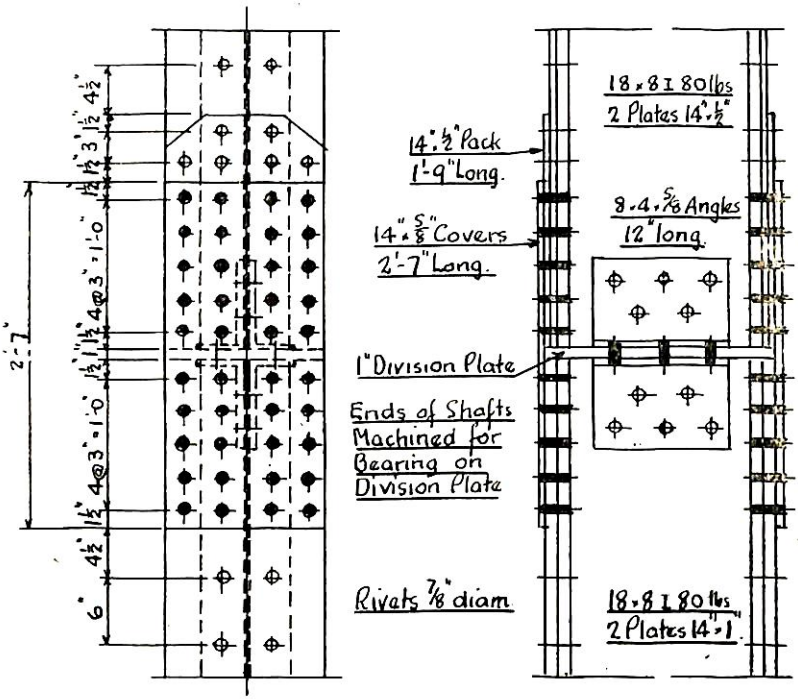

Fig. 20—Detail of Stanchion Splice.

Consider, for example, the lower length.

For cased condition $r_{yy} = 3{\cdot}6''$. $l/r = \dfrac{204}{3{\cdot}6} = 56{\cdot}7$

F_a (Table 1) = 6·25 tons per square inch.

Weight of casing = 12 tons. Total load = 438 + 12 = 450 tons.

$f_a = \dfrac{450}{79{\cdot}53} = 5{\cdot}66$ tons per sq. in. $\dfrac{f_a}{F_a} = \dfrac{5{\cdot}66}{6{\cdot}25} = 0{\cdot}90$

For uncased condition $\dfrac{f_a}{F_a} = \dfrac{5{\cdot}51}{6{\cdot}19} = 0{\cdot}89$

Thus it will be seen that the cased condition, after allowing for the weight of the casing, is slightly inferior to that of the uncased condition. This conclusion is by no means general, but is due to the fact that in this particular case the uncased section is already fairly stiff.

Cased Beams.

Clause 19d of the specification sets out the conditions under which beams may be assumed to be cased for the purpose of design. In any event the permissible stress in the compression flange must not exceed 10 tons per square inch, so that whenever a stress of 10 tons is permissible for the uncased condition no advantage is to be gained by casing, from the point of view of design. Casing therefore will only be economical when the unrestrained length of the beam is large enough to have a material effect upon the permissible stress.

Consider the case of a beam 30′-0″ span with no lateral restraint and supporting a 14″ brick wall 15′-0″ high.

Wall load = $\dfrac{30 \times 15 \times 1\frac{1}{8} \times 125}{2240}$ = 29·3 tons.

Own weight : casing, say = 6·7 tons.

36 tons.

M at centre = $\dfrac{36 \times 30 \times 12}{8}$ = 1620 tons ins.

Try 22″ × 7″ × 75 lbs. R.S.J. with two 10″ × $\frac{1}{2}$″ flange plates.

Gross area of section = 32·06 sq. ins.

Gross I_{xx} = 1677 + 1266 = 2943 ins.^{4}

$r_{xx} = \sqrt{\dfrac{2943}{32{\cdot}06}} = 9{\cdot}57''$

Gross I_{yy} = 41 + 83 = 124 ins.^{4}

$$r_{yy} = \sqrt{\frac{124}{32{\cdot}06}} = 1{\cdot}97''$$

For uncased condition $\frac{r_{xx}}{r_{yy}} = \frac{9{\cdot}57}{1{\cdot}97} = 4{\cdot}9.$ $\therefore$ $K_1 = 1{\cdot}025$

$$F_{bc} = \frac{1000 \times 1{\cdot}97}{360} \times 1{\cdot}025 = 5{\cdot}6 \text{ tons per sq. in.}$$

For cased condition permissible stress must not exceed
$5{\cdot}6 \times 1{\cdot}5 = 8{\cdot}4$ tons per sq. in.
For cased condition $r_{yy} = 0{\cdot}2\,(b+4) = 0{\cdot}2\,(10+4) = 2{\cdot}8''$

$$\frac{r_{xx}}{r_{yy}} = \frac{9{\cdot}57}{2{\cdot}8} = 3{\cdot}4. \quad \therefore \quad K_1 = 1{\cdot}4$$

$$F_{bc} = \frac{1000 \times 2{\cdot}8}{360} \times 1{\cdot}4 = > 10 \text{ tons (maximum).}$$

$\therefore$ Maximum permissible stress = 8·4 tons per sq. in.

$$Z \text{ required} = \frac{1620}{8{\cdot}4} = 193 \text{ ins.}^3 \quad Z \text{ given} = 230 \text{ ins.}^3$$

The section is slightly overstrong, but cannot be greatly reduced, as any reduction would reduce both the section modulus and the permissible stress.

Wind Resistance in Tall Buildings.

Fig. 21 sets out the general conditions for which provision has to be made. Clause 12a of the specification states that where the height of the building (that is, the height above ground level) does not exceed three times the width, special provision for stability against wind pressure is not necessary, providing the building is adequately stiffened by floors and walls. However, the procedure will be the same whatever the height, and the framework shown in Fig. 21 is convenient for explaining general principles.

Assumed wind velocity (Table 7) = 65 miles per hour.
Unit wind pressure (p) for height of 56′-0″ (Table 8) = 14·7 lbs. per sq. ft.
For convenience of calculation assume 15 lbs. per sq. ft.

Clause 12b states that the unit wind pressure shall be divided equally between the windward and leeward surfaces, acting as pressure and suction effects respectively.

Unit wind pressure on windward side = 7·5 lbs. per sq. ft.
Unit wind suction on leeward side = 7·5 lbs per sq. ft.

Division of the wind into pressure and suction effects as above means that side framing members will only take an effect of 7·5 lbs. per square foot, but the stanchions, as a whole, will take the full effect of 15 lbs. per square foot.

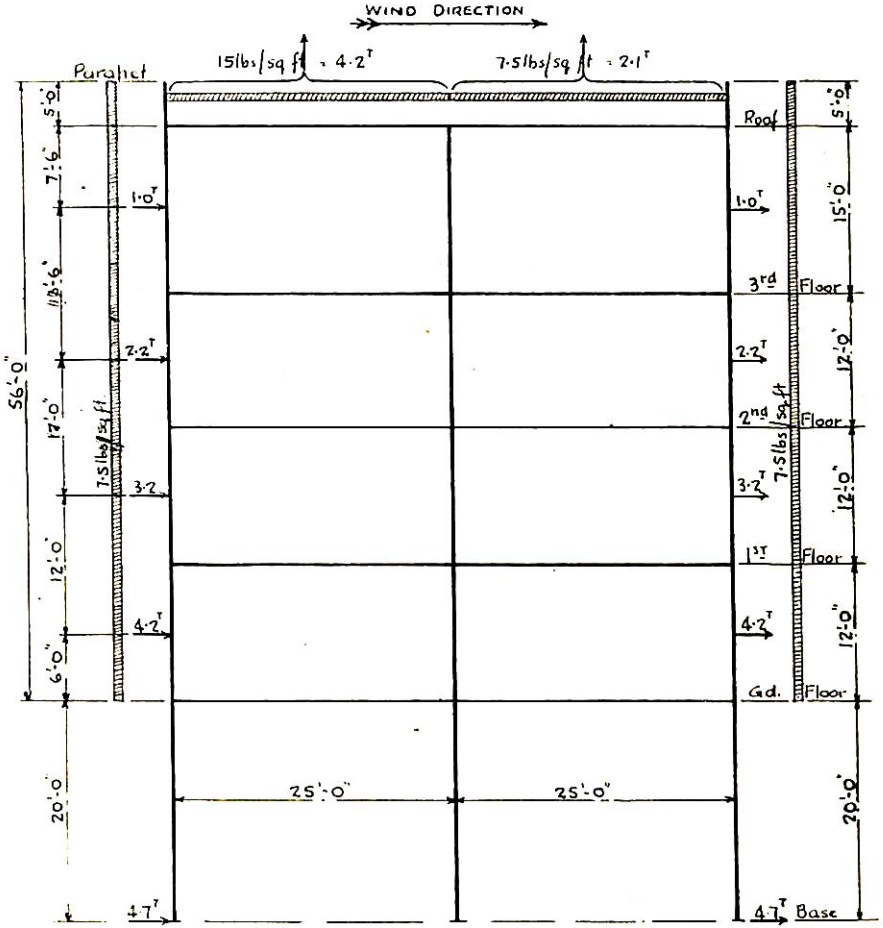

Fig. 21—Diagram of Wind Forces on Bent.

Clause 12c states that for a flat roof wind induces a suction effect on the roof amounting to $1{\cdot}0p$ on the windward half of the roof, and $0{\cdot}5p$ on the leeward half, that is in this case 15 lbs. per square foot and 7·5 lbs. per square foot respectively.

As all floors and the roof are well stiffened with beam connections, points of contraflexure in the stanchions are assumed to be midway between floors. As all stanchions have slab bases, which are incapable of taking very much bending moment, it is necessary to assume the bases act as pin ends.

Total wind above point A (per bent) $= \dfrac{25 \times 12{\cdot}5 \times 15}{2240}$

$= 2{\cdot}0^{\mathrm{T}} = 1{\cdot}0^{\mathrm{T}}$ per side.

,, ,, ,, ,, B (per bent) $= \dfrac{25 \times 26 \times 15}{2240}$

$= 4{\cdot}4^{\mathrm{T}} = 2{\cdot}2^{\mathrm{T}}$,, ,,

,, ,, ,, ,, C (per bent) $= \dfrac{25 \times 38 \times 15}{2240}$

$= 6{\cdot}4^{\mathrm{T}} = 3{\cdot}2^{\mathrm{T}}$,, ,,

,, ,, ,, ,, D (per bent) $= \dfrac{25 \times 50 \times 15}{2240}$

$= 8{\cdot}4^{\mathrm{T}} = 4{\cdot}2^{\mathrm{T}}$,, ,,

,, ,, ,, base (per bent) $= \dfrac{25 \times 56 \times 15}{2240}$

$= 9{\cdot}4^{\mathrm{T}} = 4{\cdot}7^{\mathrm{T}}$,, ,,

Suction windward half of roof (per bent) $= \dfrac{25 \times 25 \times 15}{2240}$

$= 4{\cdot}2^{\mathrm{T}}$

,, leeward ,, ,, (per bent) $= \dfrac{25 \times 25 \times 7{\cdot}5}{2240}$

$= 2{\cdot}1^{\mathrm{T}}$

Stanchion sections, designed on the basis of dead load and super loads, all consist of $18'' \times 8'' \times 80$ lbs. R.S.J. with $14'' \times 2''$ flange plates up to ground floor, $14'' \times 1''$ flange plates from ground floor to first floor, and $14'' \times \frac{1}{2}''$ flange plates from first floor to third floor, with no plates above third floor. Side stanchions have the weaker axis at right angles to the direction of the wind, and centre stanchions have the stronger axis at right angles to wind direction. It will be noticed later, when designing wind connections, that this arrangement is advantageous.

Gross moments of inertia of the stanchions are as follows :—

Bottom length—	Axis XX	=	6911 ins.^{4}
	Axis YY	=	984 ins.^{4}
	Total for bent	=	6911 + (2 × 984) = 8879 ins.^{4}
Second length—	Axis XX	=	3821 ins.^{4}
	Axis YY	=	527 ins.^{4}
	Total for bent	=	3821 + (2 × 527) = 4875 ins.^{4}
Third length—	Axis XX	=	2490 ins.^{4}
	Axis YY	=	298 ins.^{4}
	Total for bent	=	2490 + (2 × 298) = 3086 ins.^{4}
Fourth length—	Axis XX	=	2490 ins.^{4}
	Axis YY	=	298 ins.^{4}
	Total for bent	=	2490 + (2 × 298) = 3086 ins.^{4}
Top length—	Axis XX	=	1292 ins.^{4}
	Axis YY	=	69·4 ins.^{4}
	Total for bent	=	1292 + (2 × 69·4) = 1430·8 ins.^{4}

At each level in turn the building as a whole will deflect the same amount from wind effects, which means that all three stanchions in the bent will deflect the same amount. From this it follows that the share of the wind load taken by each stanchion will be proportionate to its relative stiffness, that is, the stiffer the stanchion the more wind it must take to deflect the same amount as a stanchion not so stiff. Stanchion stiffness may be measured by the factor obtained by dividing the moment of inertia by the length, and as in each section all three stanchions are the same length it follows that the proportion of wind load taken by each stanchion varies directly as the inertia about the axis at right angles to wind direction.

Individual wind loads for each stanchion length may now be calculated on the above basis.

Bottom Lengths.

Total wind force = 9·4 tons

Proportion taken by each side stanchion = $\frac{9{\cdot}4 \times 984}{8879} = 1{\cdot}04$ tons

Proportion taken by centre stanchion = 9·4 − 2·08 = 7·32 tons

Second Length.

Total wind force = 8·4 tons

Proportion taken by each side stanchion = $\frac{8{\cdot}4 \times 527}{4875} = 0{\cdot}91$ tons

Proportion taken by centre stanchion = 8·4 − 1·82 = 6·58 tons

Third Length.

Total wind force = 6·4 tons

Proportion taken by each side stanchion = $\frac{6{\cdot}4 \times 298}{3086} = 0{\cdot}62$ tons

Proportion taken by centre stanchion = $6{\cdot}4 - 1{\cdot}24 = 5{\cdot}16$ tons

Fourth Length.

Total wind force = 4·4 tons

Proportion taken by each side stanchion = $\frac{4{\cdot}4 \times 298}{3086} = 0{\cdot}43$ tons

Proportion taken by centre stanchion = $4{\cdot}4 - 0{\cdot}86 = 3{\cdot}54$ tons

Top Length.

Total wind force = 2·0 tons

Proportion taken by each side stanchion = $\frac{2 \times 69{\cdot}4}{1430{\cdot}8} = 0{\cdot}09$ tons

Proportion taken by centre stanchion = $2 - 0{\cdot}18 = 1{\cdot}82$ tons

Bending moments at each level due to wind forces may now be tabulated.

Side Stanchion Bottom Length.

M at base = Nil.
M at ground floor = $1{\cdot}04 \times 240$ = 250 tons ins.

Side Stanchion Second Length.

M at ground and first floors = $0{\cdot}91 \times 72$ = 66 tons ins.

Side Stanchion Third Length.

M at first and second floors = $0{\cdot}62 \times 72$ = 45 tons ins.

Side Stanchion Fourth Length.

M at second and third floors = $0{\cdot}43 \times 72$ = 31 tons ins.

Side Stanchion Top Length.

M at third floor and roof = $0{\cdot}09 \times 90$ = 8 tons ins.

Centre Stanchion Bottom Length.

M at base = Nil.
M at ground floor = $7{\cdot}32 \times 240$ = 1757 tons ins.

Centre Stanchion Second Length.

M at ground and first floors = $6{\cdot}58 \times 72$ = 474 tons ins.

Centre Stanchion Third Length.

M at first and second floors = $5{\cdot}16 \times 72$ = 372 tons ins.

Centre Stanchion Fourth Length.

M at second and third floors = $3{\cdot}54 \times 72$ = 255 tons ins.

Centre Stanchion Top Length.

M at third floor and roof = $1{\cdot}82 \times 90$ = 164 tons ins.

Wind moment diagrams for side and centre stanchions are given in Figs. 22 and 23.

It is now possible to draw similar wind moment diagrams for the beams connecting the stanchions together. Taking the ground floor as a typical example, the connection of the beam to the side stanchion must withstand the sum of the moments for the stanchion lengths below and above, that is, $250 + 66 = 316$ tons. ins. Similarly the connections of the beams to the centre stanchion must also withstand the sum of the moments for the stanchion lengths below and above, that is, $1757 + 474 = 2231$ tons-ins. In the latter case, however, there are two beams connecting to the stanchion, so that the connection of each beam will take half of the combined bending moment, that is, 1116 tons-ins. The beam connecting the side stanchion to the centre stanchion must have a sufficient section modulus at the two ends to withstand the wind moments of 316 and 2231 respectively, and at any point along the beam, it must be capable of withstanding—at the increased stress allowed when wind is taken into consideration—the total bending moment due to vertical load and wind at that point. Furthermore, if the end connection is of such a nature as to be able to assume the condition of fixed ends, the beam must be re-designed as fixed-ended. Generally however it is undesirable to produce deliberately a fixed-ended condition. The better procedure is to make the end connections with just sufficient strength to take the wind moments, but keeping them reasonably flexible to assume a simple-beam construction.

Fig. 24 gives wind moment diagrams for the beams at all levels, constructed in the manner described for the ground floor.

Side Stanchions.

Ground floor	=	$250 + 66$	=	316 tons ins.
First floor	=	$66 + 45$	=	111 tons ins.
Second floor	=	$45 + 31$	=	76 tons ins.
Third floor	=	$31 + 8$	=	39 tons ins.
Roof			=	8 tons ins.

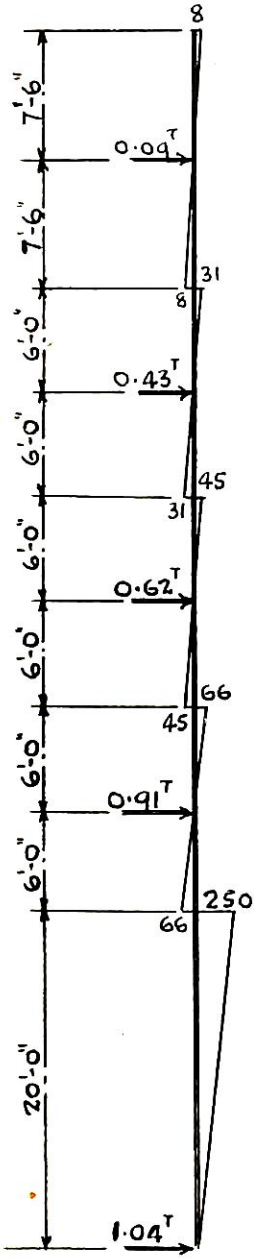

Fig. 22—Wind Moments. Side Stanchion.

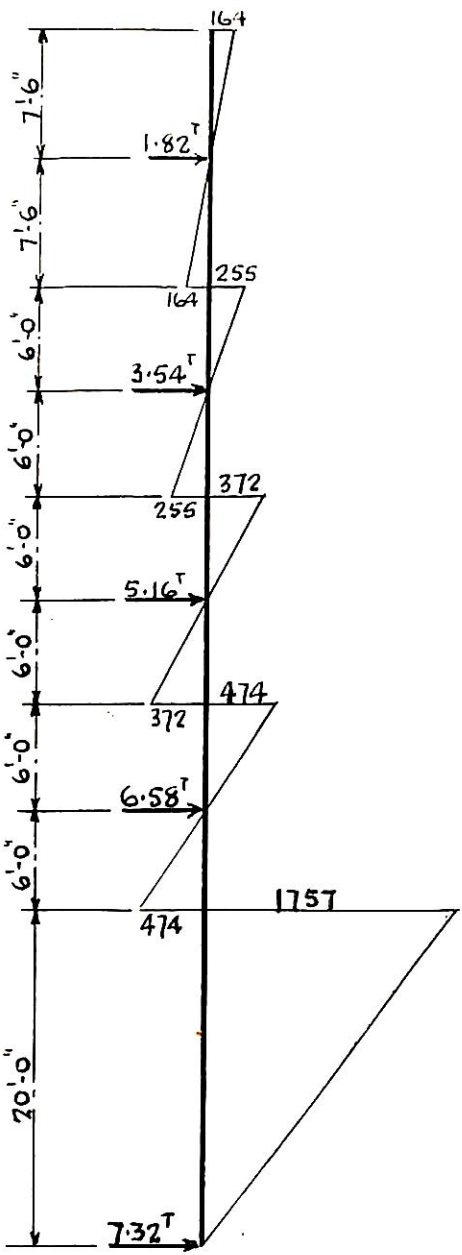

Fig. 23—Wind Moments. Centre Stanchion.

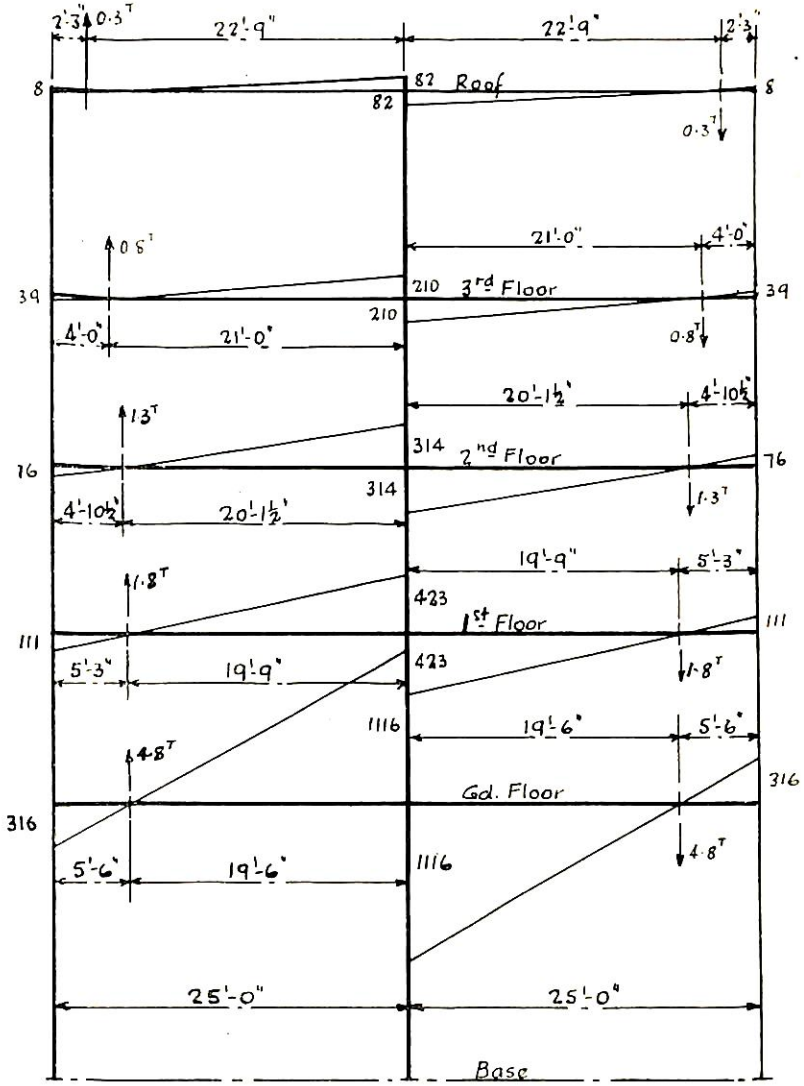

Fig. 24—Wind Moments and Shears on Beams.

Centre Stanchions.

$$\text{Ground floor} = \frac{1757+474}{2} = 1116 \text{ tons ins.}$$

$$\text{First floor} = \frac{474+372}{2} = 423 \text{ tons ins.}$$

$$\text{Second floor} = \frac{372+255}{2} = 314 \text{ tons ins.}$$

$$\text{Third floor} = \frac{255+164}{2} = 210 \text{ tons ins.}$$

$$\text{Roof} = \frac{164}{2} = 82 \text{ tons ins.}$$

From the wind moment diagrams beam shears (acting at the points of contraflexure) due to wind may now be calculated. In the leeward bay these shear forces will act downwards, and will therefore be additional to the shears at these points due to vertical load. In the windward bay the wind shears will act upward and will constitute a relief to the vertical load shears. When the beams are checked for the additional effect of wind shears the permissible stress can of course be increased by 25 per cent.

$$\text{Shear in ground floor beams} = \frac{1116}{19{\cdot}5 \times 12} \text{ or } \frac{316}{5{\cdot}5 \times 12} = 4{\cdot}8 \text{ tons}$$

$$\text{Shear in first floor beams} = \frac{423}{19{\cdot}75 \times 12} \text{ or } \frac{111}{5{\cdot}25 \times 12} = 1{\cdot}8 \text{ tons}$$

$$\text{Shear in second floor beams} = \frac{314}{20{\cdot}125 \times 12} \text{ or } \frac{76}{4{\cdot}875 \times 12} = 1{\cdot}3 \text{ tons}$$

$$\text{Shear in third floor beams} = \frac{210}{21 \times 12} \text{ or } \frac{39}{4 \times 12} = 0{\cdot}8 \text{ tons}$$

$$\text{Shear in roof beams} = \frac{82}{22{\cdot}75 \times 12} \text{ or } \frac{8}{2{\cdot}25 \times 12} = 0{\cdot}3 \text{ tons}$$

These wind shears will also slightly affect the vertical loads on the stanchions. So far as the centre stanchion is concerned the effect will be nil, as the downward shears in the leeward bay will

be counter balanced by the upward shears in the windward bay. The side stanchions in the windward bay will be relieved slightly of vertical load by the effect of wind, but the side stanchions in the leeward bay will have the vertical load increased by the effect of wind.

It is now possible to check the effects of wind moments and shears on the stanchion sections. The side stanchions have already been designed for vertical loads only in a previous example, and as a typical case the effect of wind will be checked on the bottom length of the stanchion. The leeward stanchion will be taken, as this takes additional vertical load from the beam shears.

Additional vertical load due to wind

$$= \frac{(4{\cdot}8 \times 19{\cdot}5) + (1{\cdot}8 \times 19{\cdot}75) + (1{\cdot}3 \times 20{\cdot}125) + (0{\cdot}8 \times 21)}{25} + \frac{(0{\cdot}3 \times 22{\cdot}75)}{25} = 7{\cdot}2 \text{ tons}$$

Dead load + super load = 438 tons

Total load 445·2 tons

Try 18′ × 8″ × 80 lbs. R.S.J. with two 14″ × 2″ plates as before.

Area of section = 79·53 sq. ins.

$Z_{xx} = 628{\cdot}2$ ins.^{3}. $Z_{yy} = 140{\cdot}6$ ins.^{3}. $r_{xx} = 9{\cdot}32''$. $r_{yy} = 3{\cdot}52''$.

$$\frac{r_{xx}}{r_{yy}} = \frac{9{\cdot}32}{3{\cdot}52} = 2{\cdot}6. \quad \therefore \quad K_1 = 1{\cdot}5.$$

$$l/r = \frac{204}{3{\cdot}52} = 58. \quad \therefore \quad F_a \text{ (with wind)} = 6{\cdot}19 + 25\% = 7{\cdot}74 \text{ tons per sq. in.}$$

M_{xx} (vertical loads) = 60 tons ins.
M_{yy} (vertical loads) = 65 tons ins.
M_{yy} (wind loads) = 316 tons ins.
Total M_{yy} = 65 + 316 = 381 tons ins.

$$f_{bcxx} = \frac{60}{628{\cdot}2} = 0{\cdot}1 \text{ tons per sq. in.}$$

$$f_{bcyy} = \frac{381}{140{\cdot}6} = 2{\cdot}71 \text{ tons per sq. in.}$$

Total $f_{bc} = 0{\cdot}1 + 2{\cdot}71 = 2{\cdot}81$ tons per sq. in.
$F_{bc} = 10 + 25\% = 12{\cdot}5$ tons per sq. in.

$$f_a = \frac{445{\cdot}2}{79{\cdot}53} = 5{\cdot}6 \text{ tons per sq. in.}$$

$$\frac{f_a}{F_a} + \frac{f_{bc}}{F_{bc}} = \frac{5{\cdot}6}{7{\cdot}74} + \frac{2{\cdot}81}{12{\cdot}5} = 0{\cdot}94$$

The factor 0·94 is less than unity and the section is therefore sufficient.

It will be found that if the remaining lengths of the stanchion are checked in the same way the sections already fixed up will be adequate for wind loads.

With regard to the connections of the beams to stanchions, the ground floor connection to the centre stanchion will be designed as a typical case, as the moment here is highest.

The incoming girder at this level is a plate girder with $36'' \times \frac{1}{2}''$ web, $6'' \times 6'' \times \frac{5}{8}''$ flange angles, and $14'' \times \frac{5}{8}$ plate on each flange at ends. The general construction of the connection is shown in Fig. 25. The vertical load is taken most conveniently by the web

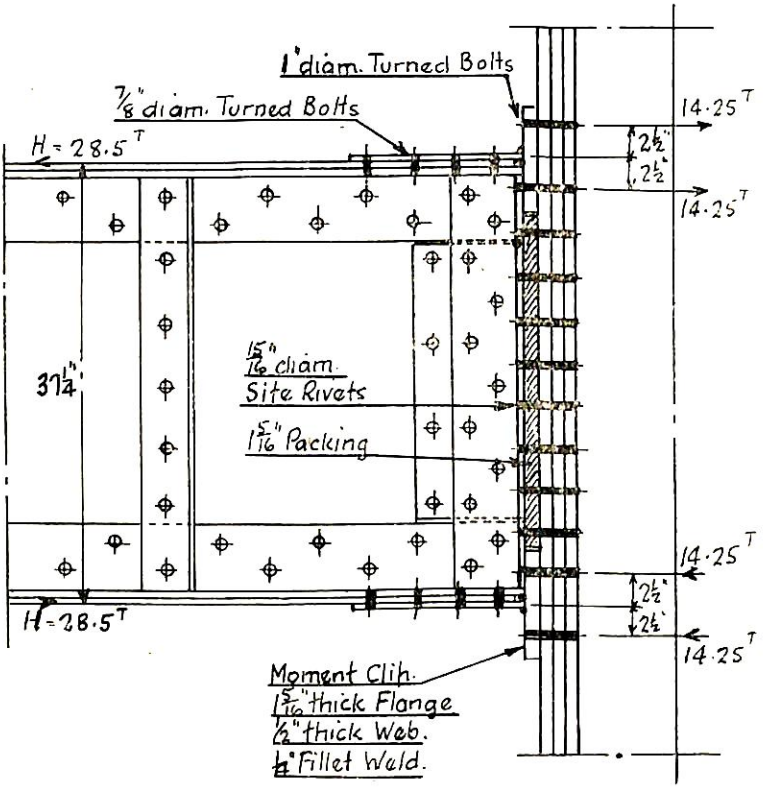

Fig. 25—Ground Floor Moment Connection.

connection, and the wind moment by the flange connections, the general principle of the latter being that the connections must be strong enough to provide a resisting couple equal to the wind moment.

Overall depth of stanchion = 1'-10".

Distance from column face to point of contraflexure on floor beam = 19'-6" − 11" = 18'-7".

Wind moment at column centre line = 1116 tons ins.

$$\therefore \text{ Wind moment at column face } = \frac{1116 \times 18{\cdot}58}{19{\cdot}5}$$

$$= 1063 \text{ tons ins.}$$

Depth of incoming plate girder = $37\frac{1}{4}$".

$$\therefore \text{ Force H at top and bottom flange } = \frac{1063}{37\frac{1}{4}} = 28{\cdot}5 \text{ tons.}$$

Bolts Through Clip Flange.

Use 1" diameter turned bolts. Area at bottom of thread = 0·554 sq. ins.

Value of bolt in tension at (6 + 25%) tons per sq. in. = 4·15 tons.

$$\text{No. of bolts required } = \frac{28{\cdot}5}{4{\cdot}15} = 7 \text{ (8 provided).}$$

Thickness of Clip Flange.

M = 14·25 × 2·5 = 35·6 tons ins.

F_b = 10 + 25% = 12·5 tons per sq. in.

$$Z \text{ required in flange } = \frac{35{\cdot}6}{12{\cdot}5} = 2{\cdot}85 \text{ ins.}^3 = \frac{Bd^2}{6}$$

Effective width of flange = 14 − (4 × 1) = 10" = B.

$$\text{Thickness required } = \sqrt{\frac{2{\cdot}85 \times 6}{10}} = 1{\cdot}31 \text{ ins.}$$

Use $1\frac{5}{16}$" thick flange.

Bolts Through Clip Web.

Use $\frac{7}{8}$" diameter turned bolts to suit girder. Area of shank = 0·60 sq. ins.

S.S. value at (6 + 25%) tons per sq. in. = 4·5 tons.

$$\text{No. of bolts required } = \frac{28{\cdot}5}{4{\cdot}5} = 7 \text{ (8 provided).}$$

Thickness of Clip Web.

Effective width = $14 - (2 \times 0{\cdot}875) = 12{\cdot}25''$.
$F_a = 9 + 25\% = 11{\cdot}25$ tons per sq. in.

$$\text{Thickness required} = \frac{28{\cdot}5}{12{\cdot}25 \times 11{\cdot}25} = 0{\cdot}21''.$$

For construction use $\frac{1}{2}''$ thick web. (Note that a minimum thickness of $\frac{3}{8}''$ is required to give a bearing value at least equal to single shear value).

Fillet Welds to Clip.

Effective length of fillet = $14''$.
Effective thickness of fillet = $0{\cdot}7t$ (where t = fillet size).
Permissible stress = $6{\cdot}5 + 25\% = 8{\cdot}1$ tons per sq, in.

$$\therefore \text{ Minimum size of fillet} = \frac{28{\cdot}5}{2 \times 14 \times 0{\cdot}7 \times 8{\cdot}1} = 0{\cdot}18''.$$

Use $\frac{1}{4}''$ fillet weld as minimum.
Vertical reaction (taken in web) = 50 tons.
S.S. value $\frac{15}{16}''$ diameter site rivets = 3·45 tons.

$$\text{No. of rivets required} = \frac{50}{3{\cdot}45} = 15 \text{ (16 provided)}.$$

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